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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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THE STRESSES IN A FREE PRISMATIC ROD UNDER A SINGLE FORCE NORMAL TO ITS AXIS

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Synopsis

The resistance of engineering structures to earthquake forces has been studied by engineers in the past but revived interest in the subject is evidenced since the Japanese earthquake of September, 1923, and the Santa Barbara earthquake of June, 1925. The Japanese seismologists and engineers, in particular, have been making intensive studies of the effect of earthquake shocks on structures for many years, and the results of their studies have been published in the *Transactions* of the Seismological Society of Japan. The writings of the late F. Omori, in particular, have become classics in earthquake literature.

Omori observed that in general tall chimneys did not overturn, nor fracture at the base, but were ruptured at a point about two-thirds of their height. To this point of rupture he applied the term, "center of percussion," which term is used in the science of physics, and is associated with the action of an impulsive force acting upon a rigid body free to move. Hence, the few writers on the subject of the resistance of engineering structures to earthquakes have assumed, in general, that, under the influence of earthquake shocks, tall, relatively slender structures act differently from low structures of relatively broad lateral dimension. Some of these writers, under the influence of Omori's statements, have assumed that the computed lateral force of the earthquake shock must be doubled for impact, in the case of high slender structures. There is evidence, also, that the distribution of stresses in such structures is not generally understood. Indeed, there is nothing in Omori's published writings to indicate that he found the distribution of bending moments and shears in a structure which exhibited the phenomenon of rotation about an instantaneous center.

In designing structures to resist earthquake forces, American engineers in general have been content to use, in lieu of an assumed earthquake horizontal force, a wind pressure which, in their opinion, was equivalent in effect to the shock of the earthquake. Some engineers, in designing high reinforced concrete chimneys, have computed the total force acting on the chimney as a whole from the assumed horizontal component of the acceleration and have

Note.—Written discussion on this paper will be closed in April, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

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then increased, or decreased, this basic acceleration along the height of the chimney to cover the unknown bending moments and shears which would result from a force causing the phenomenon of "center of percussion."

This paper sets forth, as a problem of pure mechanics, the distribution of bending moments and shears in a slender rigid prism, free to move under the action of a single impulsive force. It is found that such prism under the action of such impulsive force does tend to turn about a center of instantaneous rotation, bearing out the observations of Omori, and that a point of maximum shear occurs at such center. Formulas are given for determining the location of such center of instantaneous rotation and the distribution of shears and bending moments.

The discussion is illustrated by practical applications to specific examples, and graphs of the resulting shears and bending moments are shown.

The following analysis was undertaken by the writer as a problem in pure mechanics, without thought of its application to practical construction, but since the Santa Barbara earthquake of June, 1925, attention has been called to the fact that the formulas developed might have some bearing on the design of tall slender structures. The problem herein investigated is concerned with the stresses in a rigid prismatic rod acted upon by a single impulsive force normal to its axis when the rod has no support whatever except the yielding support of inertia. It is important to note that the analysis cannot apply as an exact quantitative method to the stresses in an engineering structure, as the elasticity of such a structure greatly reduces the magnitude of the stress. The formulas, therefore, are of qualitative rather than of quantitative value.

Let $A\,E$, Fig. 1, be a rigid prismatic rod free of all constraint. If m is its mass and l its length, its moment of inertia about an axis through the center of mass and normal to its geometric axis will be:

$$I_G = m\left(\frac{l^2}{12} + k_0^2\right) \qquad (1)$$

in which, k_0 is the radius of gyration of its areal cross-section about a parallel axis through its centroid. Now, if for such a rod, K, be the mass per unit length, $m = K\,l$, and Equation (1) becomes:

$$I_G = K\,l\left(\frac{l^2}{12} + k_0^2\right) \qquad (2)$$

Imagine a force, P, applied at any point, D (Fig. 1), a known distance x from G, the center of mass. Imagine this force to act for a very short time, a time so short that at its conclusion the angular velocity of the rod is still very small, and the normal acceleration of the mass, that is, the acceleration of particles in the direction, $E\,A$, is negligible. It is not necessary, however, that the force, P, be constant during this brief time, but it may follow any time law whatever. Such a force may be considered to act during any phase of an impulsive blow. The problem now is to determine the distribution and magnitude of the bending moments and shearing stresses along the rod as functions of P and physical constants.

First the instantaneous motion of the rod as a whole in terms of P and of the time, t, must be determined. Consider the rod hinged so as to rotate about a fixed rigid axis normal to the paper at O (Fig. 2). Then the rod is constrained, and while the force, P, is acting there will be in general a bearing reaction, B, at O parallel with P, since P has no component along the rod, since gravitation is neglected, and since the acceleration of G along the axis of the rod is so small in the short time considered as to be negligible. Taking forces to the right as positive, and counterclockwise rotation positive, from the principle of impulse and linear momentum:

$$-\int_0^t P\,dt + \int_0^t B\,dt = m v \quad\text{.................(3)}$$

v being the velocity of G at time, t. Also, from the principle of moment of momentum,

$$-\int_0^t P\,(x + r)\,dt = I_0\,\omega \quad\text{....................(4)}$$

O being at rest. The kinematic relation between v and ω is,

$$-v = -r\,\omega \quad\text{..........................(5)}$$

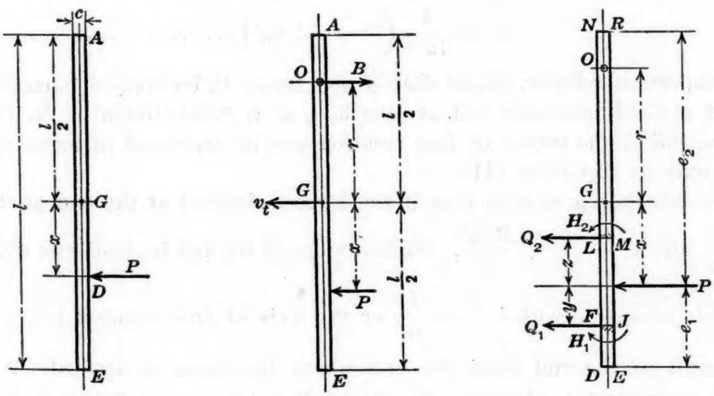

FIG. 1. FIG. 2. FIG. 3.

Remembering that $x + r$ is a constant as regards time, Equation (4) gives,

$$-\int_0^t P\,dt = \frac{I_0\,\omega}{x + r} \quad\text{.......................(6)}$$

Substituting from Equation (6) and from Equation (5) in Equation (3),

$$\frac{I_0\,\omega}{x + r} + \int_0^t B\,dt = m\,r\,\omega$$

and differentiating,

$$\frac{I_0}{x + r}\,d\,\omega + B\,dt = m\,r\,d\,\omega \quad\text{.....................(7)}$$

Equation (6), however, when differentiated gives,

$$-P\,dt = \frac{I_0}{x + r}\,d\,\omega \quad\text{......................(8)}$$

Substituting for $d\,\omega$ in Equation (7), its value from Equation (8) thus eliminating $d\,t$,

$$B = P\left(1 - \frac{m\,r\,(x + r)}{I_0}\right) \qquad (9)$$

Equation (9) shows that within the short time limit considered, B is directly proportional to P, regardless of the law of variation of P. B of course will be zero if P is zero, but even if P has any finite value, B will be zero if $m\,r\,(x + r) = I_0$. Now,

$$I_0 = I_G + m\,r^2$$

hence, if B is to be zero,

$$m\,r\,x + m\,r^2 = I_G + m\,r^2$$

$$r = \frac{I_G}{m\,x} \qquad (10)$$

If O is located at the distance, r, given by Equation (10) there will be no bearing reaction at O. The axis can then be removed, and O becomes the free axis of rotation.

Substituting in Equation (10) the value of I_G given in Equation (2);

$$r = \frac{1}{12\,x}\left(l^2 + 12\,k_0^{\,2}\right) \qquad (11)$$

This important relation shows that if any force, P, be applied normal to the axis of a rigid prismatic rod of length, l, at a point distant x ft. from G, the position of the center of free rotation can be expressed in terms of l, k_0, and x only by Equation (11).

It is interesting to note that if the force is applied at the end of the rod, $x = \frac{l}{2}$ and $r = \frac{l}{6} + \frac{2\,k_0^{\,2}}{l}$. Furthermore, if the rod is slender so that $k_0^{\,2}$ is negligible compared with l, $r = \frac{l}{6}$, or the axis of free rotation is two-thirds the length of the rod from the end where the force is applied. Also, if $x = 0$, $r = \infty$, as is obvious. Finally, if P is known as a function of t, the motion of the rod is completely determined by Equation (8). If in Equation (8) for I_0 its equivalent, $I_G + m\,r^2$, is substituted, and for r its value for a center of free rotation given in Equation (10), or $r = \frac{I_G}{m\,x}$, Equation (8) becomes,

$$d\,\omega = -\frac{P\,x}{I_G}\,d\,t$$

$$= -\frac{12\,P\,x}{K\,l\,(l^2 + 12\,k_0^{\,2})}\,d\,t \qquad (12)$$

Proceed now to determine the internal stresses. If a part of the rod be imagined as cut away by any plane normal to the axis, then the force and couple which must be applied to that section to cause it to move as indicated by Equation (12) will represent the shearing stress and bending moment in that section. Consider the stresses acting on the piece below the

plane, FJ. These will consist of a force, Q_1, and a couple, H_1, in the directions shown, Fig. 3. The linear momentum of $F\,J\,D\,E$ gives:

$$-\int_0^t Q_1\,dt = m_1\,v_1 = K\,(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right)\omega$$

The moment of momentum about the center of mass of the same piece gives:

$$-\int_0^t H_1\,dt + \int_0^t Q_1\,\frac{e_1 - y}{2}\,dt = I_G'\,\omega = K\,(e_1 - y)\left(\frac{(e_1 - y)^2}{12} + k_0^2\right)\omega$$

Differentiating the first of these latter equations, and substituting for $d\,\omega$ its value from Equation (12), there results by the elimination of $d\,t$,

$$Q_1 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right) \qquad (13)$$

This is the shearing stress in terms of y, and is a parabola.

Differentiating the second, making the same substitutions, and also substituting for Q_1 its value from Equation (13),

$$H_1 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right)\frac{e_1 - y}{2} + \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left(\frac{(e_1 - y)^2}{12} + k_0^2\right)$$

or,

$$H_1 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left[\left(r + x + \frac{e_1 + y}{2}\right)\frac{e_1 - y}{2} + \frac{(e_1 - y)^2}{12} + k_0^2\right] \qquad (14)$$

This is the bending moment in terms of y and is a cubic curve.

Again, consider the stresses in the plane, $L\,M$, acting on the section, $L\,M\,N\,R$. Call the force Q_2 and the couple H_2, in directions shown. Let z be the distance of $L\,M$ from P, and e_2 the distance of P from the top of the bar. The equations are:

$$-\int_0^t Q_2\,dt = m_2\,v_2 = K\,(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right)\omega$$

and,

$$\int_0^t H_2\,dt - \int_0^t Q_2\,\frac{e_2 - z}{2}\,dt = I_G''\,\omega = K\,(e_2 - z)\left(\frac{(e_2 - z)^2}{12} + k_0^2\right)\omega$$

Differentiating now and making the same substitutions as before, there results,

$$Q_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right) \qquad (15)$$

the shearing stress in terms of z, a parabola; and,

$$H_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - z)\left[\left(r + x - \frac{e_2 + z}{2}\right)\frac{e_2 - z}{2} - \frac{(e_2 - z)^2}{12} + k_0^2\right] \qquad (16)$$

the bending moment, a cubic curve.

Putting $y = 0$ in Equation (13) gives the shearing stress at the point of application of P necessary to give the required linear acceleration to the center of mass of the lower part; thus,

$$Q_1' = \frac{6 P x e_1}{l (l^2 + 12 k_0^2)} (2 r + 2 x + e_1) \ldots\ldots\ldots\ldots (17)$$

Again, putting $z = 0$ in Equation (15) gives the stress at P necessary to give linear acceleration to the center of mass of the upper part; thus,

$$Q_2' = \frac{6 P x e_2}{l (l^2 + 12 k_0^2)} (2 r + 2 x - e_2) \ldots\ldots\ldots\ldots (18)$$

$Q_1' + Q_2'$ should equal P, which is seen to be true, when Equations (17) and (18) are added, note being taken of the fact that $e_1 + e_2 = l$; $e_1 - e_2 = -2x$; and $r = \dfrac{l^2}{12 x} + \dfrac{k_0^2}{x}$.

Also, putting $y = 0$ in Equation (14) gives the couple in section at P which, together with Q_1', is necessary to give the entire lower part the required angular acceleration; thus,

$$H_1' = \frac{12 P x e_1}{l (l^2 + 12 k_0^2)} \left[\left(r + x + \frac{e_1}{2}\right) \frac{e_1}{2} + \frac{e_1^2}{12} + k_0^2\right] \ldots\ldots (19)$$

Finally, put $z = 0$ in Equation (16) and get the couple in the plane of P, which, together with Q_2', is necessary to give the entire upper part the required angular acceleration; thus,

$$H_2' = \frac{12 P x e_2}{l (l^2 + 12 k_0^2)} \left[\left(r + x - \frac{e_2}{2}\right) \frac{e_2}{2} - \frac{e_2^2}{12} - k_0^2\right] \ldots\ldots (20)$$

$H_1' - H_2'$ should be zero, and this is found to be true by substituting for e_1 its equal $\dfrac{l - 2x}{2}$, and for e_2 its equal $\dfrac{l + 2x}{2}$, and subtracting Equation (20) from Equation (19).

Now is there a point in the rod where either the shear or bending moment reduces to zero? Take, first, Q_1. Let,

$$Q_1 = \frac{12 P x}{l (l^2 + 12 k_0^2)} (e_1 - y) \left(r + x + \frac{e_1 + y}{2}\right) = 0$$

If $x = 0$, $r = \infty$, and the expression is indeterminate. However, if for r its equivalent, $\dfrac{1}{12 x} (l^2 + 12 k_0^2)$, is substituted and the expression reduced,

$$Q_1 = \frac{P}{l (l^2 + 12 k_0^2)} [(e_1 - y) (l^2 + 12 k_0^2 + 12 x^2 + 6 x [e_1 - y])]$$

and now putting $x = 0$,

$$Q_1 = \frac{P (e_1 - y)}{l}$$

which, in general, is not zero unless $y = e_1$. In other words, if the force, P, is applied opposite the center of mass, the shear is given by $Q_1 = \dfrac{P (e_1 - y)}{l}$,

and this is not zero except at the ends. The shear at the centroid itself is $\frac{P}{2}$, since when $x = 0$, $e_1 = \frac{l}{2}$. If x is not equal to zero, the shear is zero at the lower end in any case, since there $y = e_1$. Putting $r + x + \frac{e_1 + y}{2} = 0$, and solving for y gives $y = -(2r + 2x + e_1)$, which being negative does not fall within the limits of the piece under consideration. Hence in the lower part the shear cannot be zero except at the lower end.

Now, put,

$$Q_2 = \frac{12 P x}{l (l^2 + 12 k_0^2)} (e_2 - z) \left(r + x - \frac{e_2 + z}{2} \right) = 0$$

Here, if $x = 0$, the result is the same as in the preceding case. If x is not zero, Q_2 becomes zero when $z = e_2$, or at the upper end in any case. If, however,

$$r + x - \frac{e_2 + z}{2} = 0$$

$$z = 2r + 2x - e_2 \text{.....................} (21)$$

and this may have a definite positive value, showing that in the general case there is a point of no shear in the upper part if $e_2 < 2r + 2x$.

Consider the bending moments. Put,

$$H_1 = \frac{12 P x}{l (l^2 + 12 k_0^2)} (e_1 - y) \left[\left(r + x + \frac{e_1 + y}{2} \right) \frac{e_1 - y}{2} + \frac{(e_1 - y)^2}{12} + k_0^2 \right] = 0$$

Substitute for r its value, namely, $\frac{1}{12x}\left(l^2 + 12 k_0^2\right)$, then put $x = 0$, giving the expression for the bending moments, H_1, when P is applied opposite the center of mass,

$$H_1 = \frac{P (e_1 - y)^2}{2 l}$$

This cannot be zero unless $y = e_1$. Hence, if the force is applied opposite the center of mass the bending moments will be finite throughout except at the ends. At the center of mass itself the bending moment will be,

$$H_1 = \frac{P e_1^2}{2 l} = \frac{P l}{8}$$

Consider the expression for $H_1 = 0$ in its general form. This will be satisfied if $y = e_1$, or at the lower end. Now, put,

$$\left(r + x + \frac{e_1 + y}{2} \right) \frac{e_1 - y}{2} + \frac{(e_1 - y)^2}{12} + k_0^2 = 0$$

For any positive value of y less than e_1 the left-hand side of the equation has a positive finite value, and hence cannot be zero. It is evident, therefore, that the bending moments cannot be zero in the lower part except at the lower end.

Consider the bending moments in the upper part,

$$H_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - z)\left[\left(r + x - \frac{e_2 + z}{2}\right)\frac{e_2 - z}{2}\right.$$

$$\left. - \frac{(e_2 - z)^2}{12} - k_0^2\right] = 0$$

If the force, P, is applied opposite the center of mass, or $x = 0$, the expression reduces to the same form as did H_1. For the general case if $z = e_2$, H_2 is zero, or the bending moment is always zero at the upper end. H_2 is also zero if,

$$\left(r + x - \frac{e_2 + z}{2}\right)\frac{e_2 - z}{2} - \frac{(e_2 - z)^2}{12} - k_0^2 = 0$$

This is a quadratic equation, the final solution of which is,

$$z = \frac{1}{2}\left[3\,(r + x) - e_2\right] \pm \sqrt{\frac{9}{4}\,(e_2 - r - x)^2 + 6\,k_0^2} \quad \ldots\ldots\ldots(22)$$

There are two real roots for Equation (22), but only one of them, namely, that obtained by using the negative sign before the radical, may be less than e_2 and fall within the limit of the piece under consideration.

Consider now where the maximum shear and bending moments will come. By this is meant the true mathematical maxima, and not the values at the point of application of P which may be numerically larger than the mathematical maxima. Evidently there is no such point in the lower part, since the two functions increase continuously from zero at the lower end to their greatest values at the point of application of P. In the upper part, however, from Equation (15):

$$Q_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right)$$

Let $e_2 - z = \alpha$; then $z = e_2 - \alpha$ and $\frac{e_2 + z}{2} = e_2 - \frac{\alpha}{2}$. Call $e_2 - r - x = A$, a constant for the differentiation; then,

$$Q_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\left(A\,\alpha - \frac{\alpha^2}{2}\right)$$

Then,

$$\frac{d\,Q_2}{d\,\alpha} = -\frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(A - \alpha) = 0$$

hence,

$$A - \alpha = 0$$

or Q_2 is a maximum when $\alpha = A$, or when,

$$z = r + x \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(23)$$

opposite the center of free rotation. The magnitude of this maximum shear will be found by putting this value in Equation (15),

$$Q_2\Big]_{\max.} = -\frac{6\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - r - x)^2 \quad \ldots\ldots\ldots\ldots(24)$$

Again, in the upper part, from Equation (16),

$$H_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^{\,2})}\,(e_2 - z)\left[\left(r + x - \frac{e_2 + z}{2}\right)\frac{e_2 - z}{2} - \frac{(e_2 - z)^2}{12} - k_0^{\,2}\right]$$

Making the same substitutions,

$$H_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^{\,2})}\,\alpha\left[\left(-A + \frac{\alpha}{2}\right)\frac{\alpha}{2} - \frac{\alpha^2}{12} - k_0^{\,2}\right]$$

$$= \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^{\,2})}\left[\frac{\alpha^3}{6} - A\,\frac{\alpha^2}{2} - k_0^{\,2}\,\alpha\right]$$

Then,

$$\frac{d\,H_2}{d\,\alpha} = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^{\,2})}\left[\frac{\alpha^2}{2} - A\,\alpha - k_0^{\,2}\right] = 0$$

$$\alpha = A \pm \sqrt{A^2 + 2\,k_0^{\,2}}$$

$$z = r + x \pm \sqrt{(e_2 - r - x)^2 + 2\,k_0^{\,2}} \quad \ldots\ldots\ldots\ldots\ldots (25)$$

This gives two real values of z, but only that one obtained by using the negative sign before the radical is positive and less than e_2.

Consider a rectangular concrete column, 7 ft. long and 2 by 2 ft. in cross-section, acted upon by a force of $P = 1\,000$ lb. at right angles to the center line of one face 1 ft. from the end (Fig. 4). Then,

$$l = 7;\; P = 1\,000;\; x = 2.5;\; e_1 = 1;\; e_2 = 6;\; k_0^{\,2} = \frac{b^2}{12}$$

in which, b is the side of the square cross-sectional area, and, hence, $k_0^{\,2} = \frac{1}{3}$.

$$r = \frac{l^2 + 12\,k_0^{\,2}}{12\,x} = \frac{49 + 4}{30} = 1.766 \text{ ft.};\; \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^{\,2})} = 80.9$$

$$r + x = 4.266 \text{ ft.}$$

$$Q_1 = 80.9\,(1 - y)\left(4.766 + \frac{y}{2}\right)$$

$$H_1 = 80.9\,(1 - y)\left[\left(2.383 + \frac{y}{4}\right)(1 - y) + \frac{(1 - y)^2}{12} + \frac{1}{3}\right]$$

$$Q_2 = 80.9\,(6 - z)\left(1.266 - \frac{z}{2}\right)$$

$$H_2 = 80.9\,(6 - z)\left[\left(0.633 - \frac{z}{4}\right)(6 - z) - \frac{(6 - z)^2}{12} - \frac{1}{3}\right]$$

These curves are plotted on Fig. 4.

It will be noted that zero shear in the upper part comes at,

$$z = 2r + 2x - e_2$$
$$= 8.533 - 6 = 2.533 \text{ ft.}$$

Maximum shear comes at $z = r + x$, or at $z = 4.2666$ ft., opposite O. The shear at this point will be $Q = 121.4$ lb.

Zero bending moment in the upper part comes at,

$$z = \frac{1}{2}\,[3\,(r + x) - e_2] - \sqrt{\frac{9}{4}\,(e_2 - r - x)^2 + 6\,k_0^2}$$

$$= \frac{3}{2} \times 4.266 - 3 - \sqrt{6.76 + 2} = 0.440 \text{ ft.}$$

Maximum bending moment comes at,

$$z = r + x - \sqrt{(e_2 - r - x)^2 + 2\,k_0^2}$$

$$= 4.266 - \sqrt{3 + \frac{2}{3}} = 2.352 \text{ ft.}$$

The bending moment at this point is $H = -376.5$.

The formulas and relations developed thus far can be simplified to a considerable extent if the prism under consideration is so slender as to make $12\,k_0^2$ negligible in comparison with l^2. In other words, the moment of inertia of a slender rod about an axis normal to its own axis and through the center of mass is $I_G = m\,\frac{l^2}{12}$. It must be observed, however, that in developing the expressions for bending moments, the moment of inertia of a part of the prism such as $F\,J\,D\,E$ (Fig. 3) enters into the equation for the bending moment in the section, $F\,J$. Calling this I_G' and using the simplified expression,

$$I_G' = m_1\,\frac{(e_1 - y)^2}{12}$$

This will not hold with the same degree of accuracy as y approaches e_1. The simplified expressions for bending moments, therefore, are not exact toward the ends. Since the stresses there are small this fault may be overlooked, and the results obtained are practically exact.

The simplified formulas are ($k_0^2 = 0$):

$$I_G = \frac{K\,l^3}{12}; \quad r = \frac{l^2}{12\,x}; \quad d\,\omega = -\frac{12\,P\,x}{K\,l^3}\,dt$$

$$Q_1 = \frac{12\,P\,x}{l^3}\,(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right) \qquad (13a)$$

$$H_1 = \frac{6\,P\,x}{l^3}\,(e_1 - y)^2\left(r + x + \frac{2\,e_1 + y}{3}\right) \qquad (14a)$$

$$Q_2 = \frac{12\,P\,x}{l^3}\,(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right) \qquad (15a)$$

$$H_2 = \frac{6\,P\,x}{l^3}\,(e_2 - z)^2\left(r + x - \frac{2\,e_2 + z}{3}\right) \qquad (16a)$$

Now putting y and z equal to zero,

$$Q_1' = \frac{6\,P\,x\,e_1}{l^3}\,(2\,r + 2\,x + e_1) \qquad (17a)$$

$$Q_2' = \frac{6\,P\,x\,e_2}{l^3}\,(2\,r + 2\,x - e_2) \qquad (18a)$$

$$H_1' = \frac{2\,P\,x\,e_1^2}{l^3}\,(3\,r + 3\,x + 2\,e_1)\ldots\ldots\ldots\ldots\ldots(19a)$$

$$H_2' = \frac{2\,P\,x\,e_2^2}{l^3}\,(3\,r + 3\,x - 2\,e_2)\ldots\ldots\ldots\ldots\ldots(20a)$$

The zero shear in the upper part comes at,

$$z = 2\,r + 2\,x - e_2\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(21a)$$

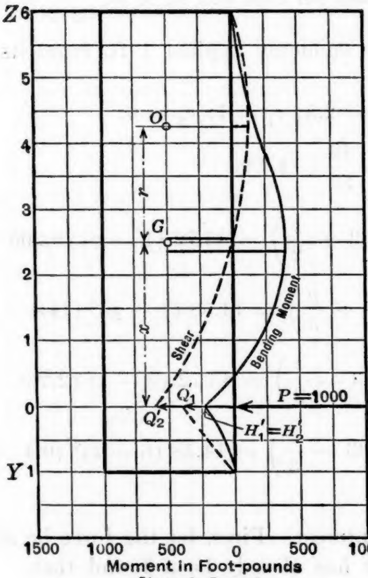

FIG. 4.

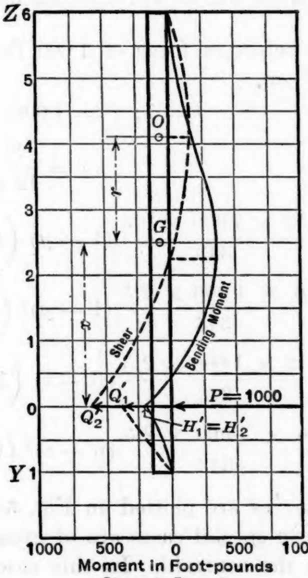

FIG. 5.

The zero bending moment in the upper part comes at,

$$z = 3\,r + 3\,x - 2\,e_2\ldots\ldots\ldots\ldots\ldots\ldots\ldots(22a)$$

The maximum shear in the upper part comes at,

$$z = r + x\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(23a)$$

or opposite the center of free rotation as in the general case. The value of this maximum shear will be,

$$Q_2\Big]_{\max.} = -\frac{6\,P\,x}{l^3}\,(e_2 - r - x)^2\ldots\ldots\ldots\ldots\ldots(24a)$$

The maximum bending moment in the upper part comes at,

$$z = 2\,r + 2\,x - e_2\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(25a)$$

opposite the point of zero shear. The value of this maximum bending moment will be,

$$H_2\Big]_{\max.} = -\frac{8\,P\,x}{l^3}\,(e_2 - r - x)^3\ldots\ldots\ldots\ldots\ldots(26a)$$

In order to illustrate the use of these approximate formulas, consider a rod of steel 7 ft. long and 3 in. in diameter (Fig. 5). In this case,

$$k_0^2 = \frac{r^2}{4} = \frac{1.5^2}{4 \times 144} = 0.00391;\ 12\ k_0^2 = 0.0469$$

which is small enough compared with $l^2 = 49$ to be neglected. For a piece of the bar ½ ft. long, however, $12\ k_0^2$ is about 19% of l^2. The weight per foot of such a rod will be 24 lb., so that its mass per foot will be $K = \frac{24}{32.2} = 0.7455$. Let, as before, a force of 1 000 lb. be suddenly applied 1 ft. from its lower end. Then,

$$l = 7;\ P = 1\,000;\ x = 2.5;\ e_1 = 1;\ e_2 = 6;$$

$$r = \frac{l^2}{12\ x} = \frac{49}{30} = 1.633$$

$$Q_1 = \frac{12 \times 1\,000 \times 2.5}{343} (1 - y) \left(4.633 + \frac{y}{2}\right) = 43.72\ (1 - y)\ (9.266 + y)$$

$$H_1 = \frac{6 \times 1\,000 \times 2.5}{343} (1 - y)^2 \left(4.8 + \frac{y}{3}\right) = 14.58\ (1 - y)^2\ (14.4 + y)$$

$$Q_2 = \frac{12 \times 1\,000 \times 2.5}{343} (6 - z) \left(1.133 - \frac{z}{2}\right) = 43.72\ (6 - z)\ (2.266 - z)$$

$$H_2 = \frac{6 \times 1\,000 \times 2.5}{343} (6 - z)^2 \left(0.133 - \frac{z}{3}\right) = 14.58\ (6 - z)^2\ (0.4 - z)$$

These curves are plotted on Fig. 5.

Certain special cases are of great interest. First, let the force be applied opposite the centroid. In this case it has already been found that,

$$Q_1 = \frac{P\ (e_1 - y)}{l}$$

and,

$$H_1 = \frac{P\ (e_1 - y)^2}{2\ l}$$

Also, Q_2 and H_2 will be the same. Taking the same data as in the last numerical case, except that,

$$e_1 = 3.5;\quad x = 0;\quad r = \infty$$

$$Q_1 = \frac{1\,000\ (3.5 - y)}{7} = 142.9\ (3.5 - y)$$

$$H_1 = \frac{1\,000\ (3.5 - y)^2}{14} = 71.4\ (3.5 - y)^2$$

These curves are plotted on Fig. 6.

Let the force, P, be applied at the extreme lower end. Then, in the same case, still using the approximate formulas,

$$x = \frac{l}{2};\quad e_1 = 0;\quad e_2 = l;\quad r = \frac{l}{6}$$

Q_1 and H_1 disappear, and,

$$Q_2 = \frac{12\,P\,l}{2\,l^3}\,(l - z)\left(\frac{l}{6} + \frac{l}{2} - \frac{l + z}{2}\right)$$

$$= \frac{P}{l^2}(l - z)(l - 3\,z)$$

$$H_2 = \frac{6\,P\,l}{2\,l^3}\,(l - z)^2\left(\frac{l}{6} + \frac{l}{2} - \frac{2\,l + z}{3}\right)$$

$$= -\frac{P}{l^2}\,(l - z)^2\,z$$

Substituting numerical values:

$$Q_2 = 20.4\ (7 - z)\,(7 - 3\,z),$$

$$H_2 = 20.4\ (7 - z)^2\ z.$$

These curves are plotted on Fig. 7.

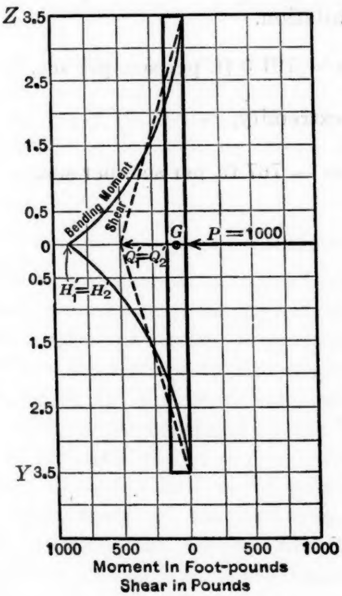

FIG. 6.

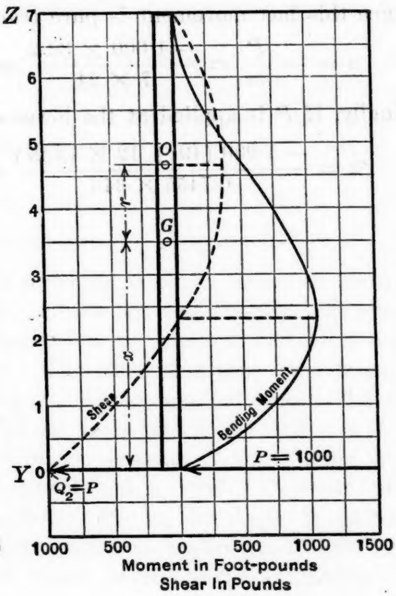

FIG. 7.

It may be noted that instead of the impressed force, P, the acceleration of the point of application may be given. Equation (12) is,

$$\frac{d\,\omega}{dt} = -\frac{12\,P\,x}{K\,l\,(l^2 + 12\,k_0^{\,2})}$$

but,

$$(r + x)\,\frac{d\,\omega}{dt} = \frac{d\,v}{dt} = a$$

the acceleration of the point of application of P. Then,

$$P = -\frac{K\,l\,(l^2 + 12\,k_0^2)\,a}{12\,x\,(r + x)} = \frac{-K\,l\,(l^2 + 12\,k_0^2)\,a}{l^2 + 12\,k_0^2 + 12\,x^2}$$

If $12k_0^2$ is neglected, these become,

$$P = \frac{-K\,l^3\,a}{12\,x\,(r + x)} = \frac{-K\,l^3\,a}{l^2 + 12\,x^2}$$

In the numerical instance last cited where $12\,k_0^2$ is neglected, if the force, P, be applied at a point 1 ft. from the lower end, then,

$$a = \frac{-1\,000\,(49 + 12 \times 6.25)}{0.7455 \times 343} = -\,485 \text{ ft. per sec. per sec.}$$

If opposite the centroid,

$$a = \frac{-1\,000\,(49)}{0.7455 \times 343} = -\,191.5 \text{ ft. per sec. per sec.}$$

Since this last movement is pure translation,

$$a = -\frac{P}{m} = \frac{-1\,000 \times 32.2}{7 \times 24} = -\,191.5 \text{ ft. per sec. per sec.}$$

Finally, if P is applied at the lower extremity,

$$a = \frac{-1\,000\,(49 + 12 \times 12.27)}{0.7455 \times 343} = -\,767 \text{ ft. per sec. per sec.}$$

TESTS OF MANHOLE COVERS

By T. J. Corwin, Jr.,* Assoc. M. Am. Soc. C. E.

Synopsis

This paper is a résumé of a report by the Division of Civil Engineering of the Pacific Gas and Electric Company, in which the theoretic stresses and deflections of manhole covers are compared with the values obtained by test.

A series of tests was made to determine the merits of the different designs and also to determine if a saving could be effected by using cast steel instead of cast iron.

This investigation was brought about by a request of the Division of Electric and Steam Distribution of the Pacific Gas and Electric Company for a change in the design of the manhole covers that would result in a saving in the initial cost and in an increased strength to conform with present traffic requirements.

Introduction

The only information regarding previous tests on manhole covers was obtained through engineering magazines. In one test the ultimate strengths of three types of covers were obtained, but no attempt had been made to determine the stresses in the covers. The other test based the strength of the cover on its ability to withstand a suddenly applied load. The magnitude of the force depended on the height from which the load was dropped.

The strength of the covers used in the systems of the Pacific Gas and Electric Company had never been determined by test. However, at present, replacement of the cast-iron covers due to breakage is exceedingly small and breakage occurs only under unusual traffic conditions. In the past, when a cast-iron cover failed under severe traffic conditions in the East Bay District of the Company, the cover was replaced by one of cast steel of the same design. The cast-steel covers have been satisfactory, so it can be assumed that their ultimate strength is sufficient for existing traffic conditions. The cast-steel ventilating cover has been subjected to the most severe traffic conditions but no failures have been reported.

Analysis of the Design of the Old Covers

After studying the design of the old ventilating cover, Fig. 1 (No. 1-1149), it appeared evident that a reduction could be made in the weight, for the

Note.—Written discussion on this paper will be closed in April, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

* Asst. Engr., Pacific Gas & Elec. Co., San Francisco, Calif.

two systems of ribs were not of equal strength. These covers were of cast steel and weighed 630 lb. The ribs were of constant thickness for the full width of the cover, whereas a varying thickness could be used in order to reduce the weight.

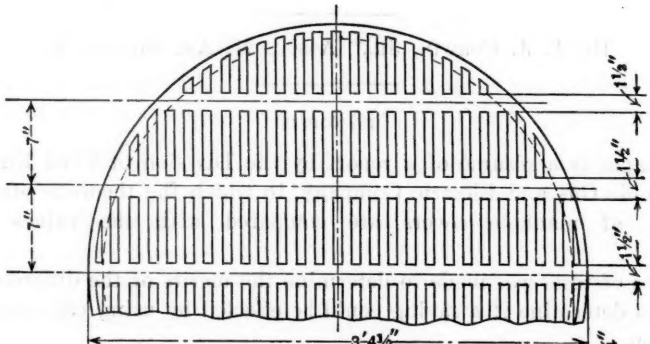

FIG. 1.—OLD VENTILATING COVER (No. 1—1149).

A study of the solid cover, Fig. 2 (No. 31086), did not indicate a surplus of material and unless radial ribs were detrimental to the strength, no reduction in the weight could be made, assuming a flat plate would support no greater load than theoretical computations indicated.

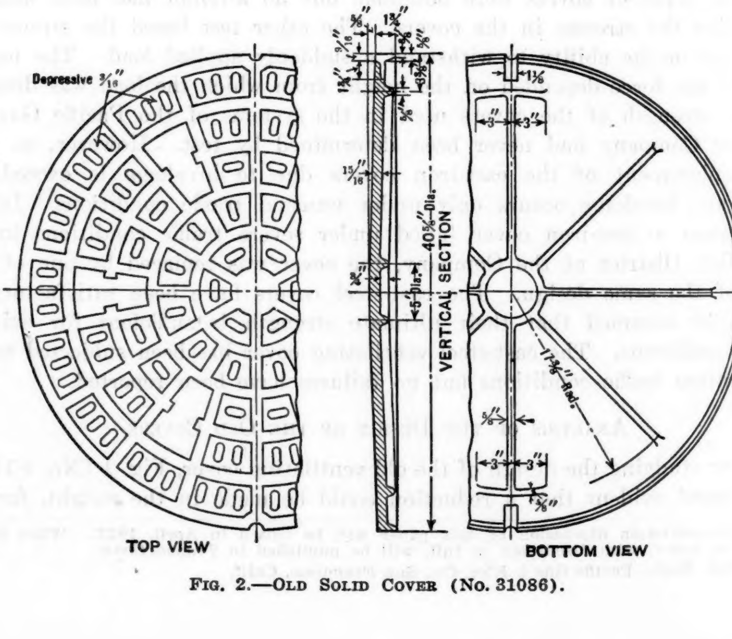

FIG. 2.—OLD SOLID COVER (No. 31086).

Re-Design of the Covers

A load of 15 000 lb. plus an impact factor of one-third was assumed as the working load. On this basis the covers were designed for an ultimate load of 70 000 lb. In order to simplify the theoretical computations this load was applied through a cylindrical loading block instead of in the manner it would be applied by the wheel of a truck.

The weight of the cover was a factor that was kept constantly in mind. However, the minimum thickness of casting advisable was the factor that really established the final weight, after the strength requirements had been satisfied.

Ventilating Cover (*Fig. 3*).—The re-design of the ventilating cover (No. 25370) followed two requirements shown by the old cover: The area of the openings and the height of the supporting ring. The main ribs were spaced so that the design load would be carried by three ribs. From the symmetry of the design one-half the load will be taken by each set of ribs. The spans of the three center ribs for the purpose of analysis were taken as equal. The ribs were made equal in area. To compute the bending moment the load was considered as concentrated at a point equal to the average radius of the loading block, from the center of the cover. The small grate bars were of sufficient strength to transmit the load to the main ribs.

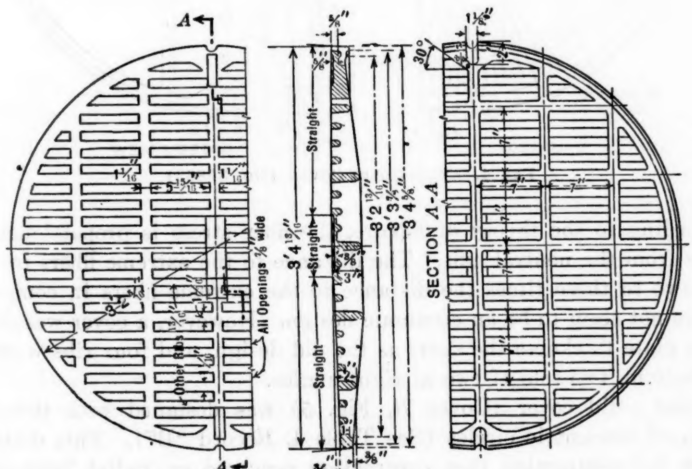

Fig. 3.—New Ventilating Cover (No. 25370), Cast Steel.

Solid Cover (*Fig. 4*).—Assuming that the radial ribs were not the most efficient type of reinforcing, only two other systems of ribs could be used. One system was in the form of concentric circular ribs, the other a system of rectangular ribs similar to the ones used in the ventilating cover.

The re-design of this cover (No. 36863) required considerable study. With the tensile strength of the metal less than one-half its compressive

strength, it appeared evident that all ribs should be on the top or compressive side for a cover of minimum weight and maximum strength. Deep ribs on the top would be objectionable to traffic and pedestrians. The cost of filling in between the ribs with asphalt or cement would probably more than offset the saving in weight of metal. The cover could not be domed for it would be objectionable to traffic, and a slight crown will not increase the strength. Placing ribs on the bottom would seem to defeat the purpose in view when adding them.

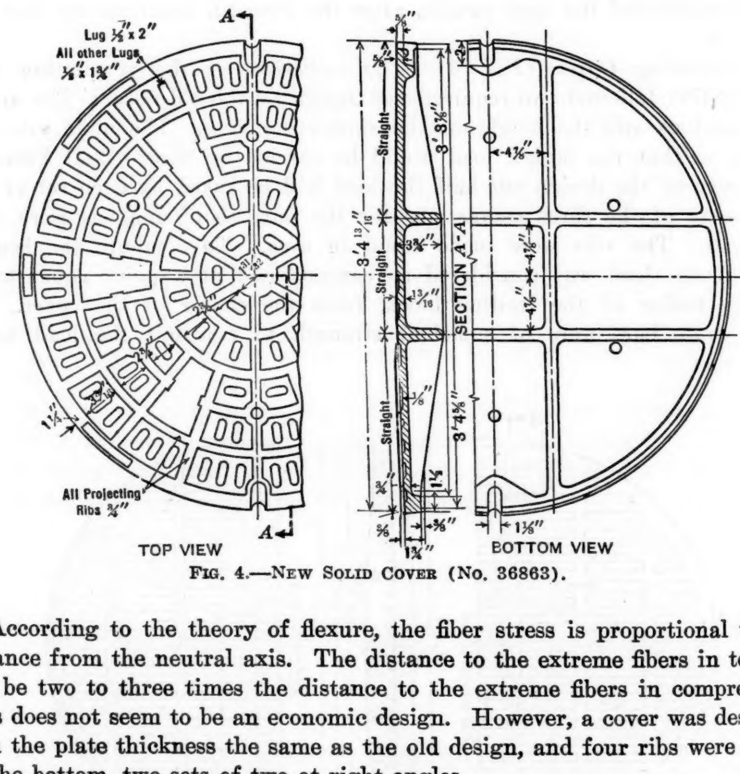

Fig. 4.—New Solid Cover (No. 36863).

According to the theory of flexure, the fiber stress is proportional to the distance from the neutral axis. The distance to the extreme fibers in tension will be two to three times the distance to the extreme fibers in compression. This does not seem to be an economic design. However, a cover was designed with the plate thickness the same as the old design, and four ribs were added on the bottom, two sets of two at right angles.

Another solid cover (Cover *B*, Fig. 5) was designed with the ribs in the form of concentric circles (See Table 4, Record "B"). This design was based on the assumption that contraction occurred on radial lines and the stresses might be equalized in this manner.

Procedure for Test and Equipment

A 300 000-lb. wheel-press was used for the test. To insure an equal distribution of the load a special casting was designed to transmit the load from the manhole frame to the frame of the wheel-press.

The tests were made under conditions approximating those that will be obtained in the field on all new installations. The lower side of the cover

and its seat in the frame were machined to insure a uniform bearing. This will prevent the rattling of the cover in the frame and also some breakage due to unequal bearing. The lower side of the manhole frame and the surface of the bearing plate were machined so that the cover would be normal to the ram of the press.

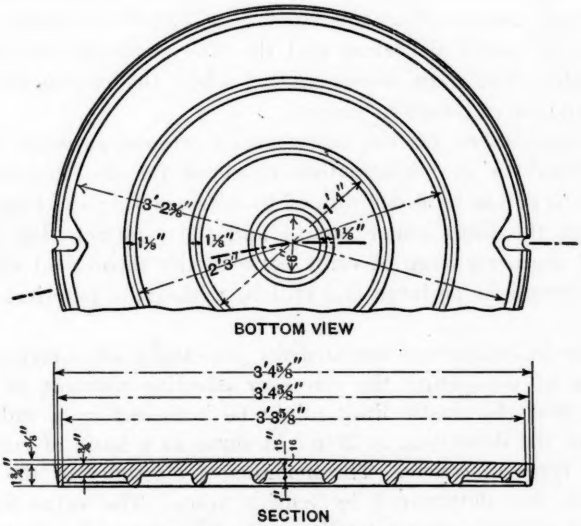

FIG. 5.—SOLID RIBBED COVER.

The extensometers were made specially for the test. The main consideration was to obtain an extensometer that could be clamped to the ribs of the cover. Two Ames dials indicating the deformations to the nearest 0.001 in. were used with each extensometer. The extreme fiber deformation is readily computed from the deformations indicated by the two dials and their respective distances from the extreme fiber. The extensometer rods passed through holes in the manhole frame, placing the dials on the outside of the frame so that the deformation indicated on the dials could be easily recorded.

The center deflection of the cover with respect to the frame of the wheel-press was determined with an Ames dial. This method included the deflection of the bearing plate. Considering the relative moments of inertia of the two sections, it is apparent that the deflection of the loading block is negligible compared with the center deflection of the cover.

A 6-in. gauge length was used for all but one test. The loading block was designed so that the bending moment would be constant for the center 7 in. of the cover. Fig. 6 shows the complete equipment.

Two extensometers were connected at right angles on the covers. From the symmetry of the design and the fact that all surfaces were machined, one would assume that the deformations at right angles would be equal. This assumption was verified in the tests.

The pump for the hydraulic press was not equipped with an accumulator, and, therefore, it was difficult to obtain a steady increase in the pressure. In many cases, it was necessary to reduce the load a few thousand pounds in order to obtain the desired value. This was particularly true at the lower load intensities, for the pressure built up rapidly.

The smallest practical increment of load was one division on the 3 000-lb. Ashton pressure gauge. This was equal to a 3 180-lb. increment of load on the cover. The center deflection and the fiber deformations were recorded simultaneously. The dials were removed when the elastic limit had been passed and the cover tested to failure.

Zero readings taken on the extensometer were apparently in error and not proportional to the deformation expected for the first increment of load. The zero points were determined by extrapolating the load-deformation curves. With the short gauge length and the relatively low elastic limit of cast steel the percentage of error between the theoretical stress and the measured value might be large and still be within the practical limit of the equipment.

The center deflection was recorded for practically all covers, but not with the intention of computing the stress or effective moment of inertia. It was thought that the elastic limit might be indicated by a sudden increase in the rate of the deflection, or it might serve as a basis of comparison for the different types of covers. The modulus of elasticity of the metals used for the covers was determined by tension tests. The value for cast steel was in excess of the value normally used. The value for "semi-steel" was approximately the same as that normally used for cast iron.

Comparison of Theoretical and Measured Stresses

Ventilating Cover (Fig. 3).—The bending moment and the section modulus were computed and the corresponding stress was determined. The unit stress was also computed from the deflection data obtained during the test. The figures in Table 1 show the variation in the stresses. One would expect the theoretical stress to be higher than the measured stress, for the ribs on the sides of the center ribs must also contribute to the strength of the cover. The difference in these stresses should become greater as the load increases, for the deflection of the center ribs will cause the side ribs to carry a larger part of the load. Some variation is also due to the fact that the moment of inertia was assumed as constant in the computations. The ribs have a variable moment of inertia, but as the moment of inertia does not vary directly with relation to the span length, the exact formula was not used. The true stresses should fall between these assumed conditions. The deflection is greater with this section than with ribs of constant section, therefore, the stress computed from the measured deflection is probably 10 to 20% higher than the actual value.

Cover *A*, (Fig. 3) with an exceptionally low elastic limit, did not fail until the load was in excess of 100 000 lb. At this load it had dished in about 1½ in. When the load was released the cover had a permanent set of 1 in.

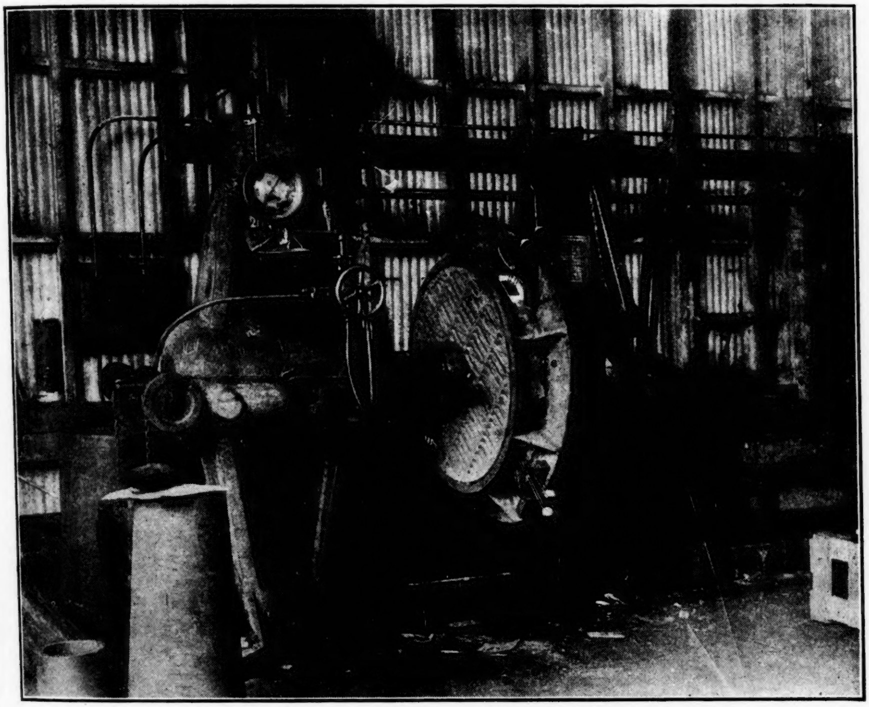

Fig. 6.—Apparatus for Testing Manhole Covers.

Fig. 7.—Typical Fractures of Manhole Covers.

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The only indication of fracture was at the junction of the main ribs and the supporting ring.

Cover No. 11 was tested to 69 000 lb. At this load the deformation of the extreme fibers was increasing rapidly. The cover had a permanent set of ⅜ in., indicating that the ultimate load was not greatly in excess of the maximum test load.

Test bars cut from Cover *A* showed that the ribs contained many flaws. With a casting of this type it would be difficult to obtain metal entirely free from flaws. The yield point determined by testing the cover, by tension tests, and as specified by the steel foundry, all agree satisfactorily.

Table 1 shows a comparison of the stress computed from the theoretical moment of inertia, the effective moment of inertia as computed from the center deflection, and the values measured on the two sets of ribs.

TABLE 1.—Steel Cover, Ventilating Type, Test No. 11.

Load, in pounds.	Moment, in inch-pounds.	Unit Stresses, in Pounds per Square Inch.			
		From theoretical *I*.	From center deflection.	Ribs.	
				Horizontal.	Vertical.
9 500	72 400	10 000	12 300	8 750	10 000
15 860	121 000	16 700	20 200	13 750	15 000
22 200	169 000	23 400	26 600	17 500	20 000
28 580	217 500	29 600	32 200	23 750	25 000
34 310	261 500	35 400	38 700	25 000	30 000
40 350	307 500	41 800	45 000	35 000	40 000
46 710	356 500	48 500	52 400	43 750	45 000
53 400	407 000	......	......	......	......
60 070	458 000	......	......	......	......
63 570	485 000	......	......	......	......
66 750	509 500	......	......	......	......

Solid Covers.—At the outset, trial computations indicated that empirical formulas would not give logical values. Three analyses were used in computing the stresses. The results obtained by two analyses derived by different methods are tabulated with the measured values.

Cast-Steel Covers

These covers Fig. 2 (No. 31086) were taken from service in the East Bay District. The extensometers could not be attached in the regular 7-in. position for a constant bending moment, so a 9-in. gauge length was used, which undoubtedly affects the accuracy of the results.

The measured values are apparently in error for the first part of the test; however, at the elastic limit the measured stresses are approximately the values expected for cast steel and more nearly agree with the theoretical computations. The results of the test on Cover No. 16 are shown in Table 2. It will be noted that for the conditions of this test (applicable only to this design) the analysis by W. H. Burr, M. Am. Soc. C. E., gives stress intensities equal numerically to the load, in pounds.

TABLE 2.—STEEL COVER, SOLID TYPE, TEST No. 16.

Load, in pounds.	Center Deflection, in Thousandths of an Inch.		Unit Stresses, in Pounds per Square Inch.		
	Measured.	Computed.	Fuller and Johnson computations.*	Burr computations.†	Measured.
6 320	5	60	8 900	6 320	3 260
9 600	59	91	13 550	9 600	3 400
12 680	72	121	17 900	12 680	6 860
16 500	97	157	23 250	16 500	20 400
19 040	106	182	26 850	19 040	20 500
22 220	125	212	31 300	22 220	27 400
25 400	146	242	35 800	25 400	30 900
27 950	172	266	39 400	27 950	37 900
31 130	192	297	43 800	31 130	34 900
31 310	214	327	48 400	34 310	44 900
37 490	245	357	52 900	37 490	44 900
40 350	253	374	56 800	40 350	48 500
43 530	...	...	61 300	43 530	62 500
46 710	...	...	66 000	46 710	62 500
49 900	...	...	......	49 900	59 900
53 400	...	...	......	53 400	63 100
56 890	...	...	......	56 890	56 500
60 000	...	...	......	60 000	54 900

* "Applied Mechanics," Vol. II.

† "The Elasticity and Resistance of Materials of Engineering," 7th Edition.

"Semi-Steel" New Standard Covers (Fig. 4).

1.—The results obtained by test agree fairly well with the theoretical values; the maximum fiber stress in tension agrees with the ultimate tensile strength of the metal. It was very difficult to obtain the fiber deformation with the equipment at hand and the tests had to be made a number of times before the results proved satisfactory.

2.—*Concentric Circular Ribs.*—As the covers did not meet the requirements for ultimate strength, no attempt was made to check the stresses in the plate.

3.—*Old Standard.*—The ultimate strength was the only value desired for this cover, Fig. 2 (No. 31086).

Table 3 shows the comparison of the stresses for the cover shown on Fig. 4.

TABLE 3.—SEMI-STEEL COVER, SOLID TYPE, TEST No. 12.

Gauge, in pounds per square inch.	Load, in pounds.	Moment, in inch-pounds.	Unit Stresses, in Pounds per Square Inch.		
			Fuller and Johnson computations.	Burr computations.	Measured.
	0	0	0	0	0
165	9 500	48 400	3 250	2 810	2 800
265	15 860	81 000	5 430	4 710	4 060
360	21 900	111 800	7 500	6 500	5 310
460	28 260	144 500	9 660	8 400	5 310
555	34 310	175 000	11 750	10 150	7 510
650	40 350	205 000	13 800	11 900	10 600
750	46 720	238 000	16 000	13 750	11 900
855	53 400	272 000	18 250	15 800	13 150

Discussion of the Results of the Test

With the design adopted by the Pacific Gas and Electric Company for the new standard, failure occurred either along one of the ribs or at an intersection of the ribs. The intersections of the ribs were readily located by depressions in the top of the cover. After failure had occurred, inspection showed that the properties of the metal at the intersection were different from those of the main mass, by a marked change in the color. All covers failed in practically the same manner. Considering the relative weakness of the ribs in tension, one would expect the cover to fail across the diameter at right angles to a set of ribs. This was almost the case, except for the apparent weakness at the intersection of the ribs, for in each cover failure occurred through at least two of these points. The first cover just reached the design strength required. Other tests indicated that the cover was capable of resisting an ultimate load of 100 000 lb. and that failure at 69 000 lb. was due to a flaw in the ribs of the specimen.

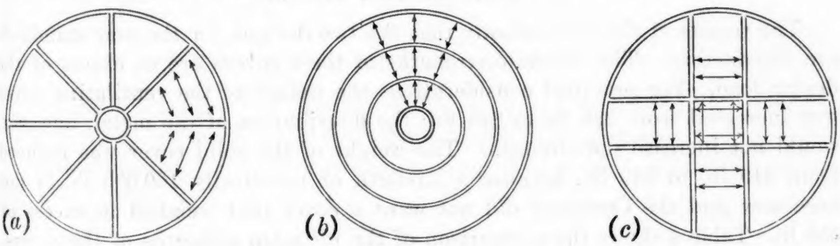

FIG. 8.

The cover with concentric ribs failed principally on radial lines, except for encircling the small center rib. With the old standard cover, failure occurred almost without exception adjacent to a rib of the cover. The first crack appeared along a rib and the rapid deflection that followed was probably the reason for failure occurring at such a small load. Fig. 7 shows the typical failures for the three different designs.

A discussion of the distribution of stress in a circular plate is essential, however, in order to justify the merits of the different designs. The stresses in a flat circular plate are tensile and compressive stresses along radial and tangential lines and direct shearing stresses. The shearing stress will not be considered for it will never be parallel to the main stresses caused by contraction. Considering the cover with radial ribs: When the metal cools it obviously solidifies between the ribs first, and the tendency will be to set up initial stresses between the ribs, Fig. 8(*a*). These stresses should be normal to the bisector of the angles formed by the ribs. Analyzing the stresses in the plate, it is found that the radial stress at the circumference is zero, but the radial and tangential stresses are equal and a maximum at the center. The tangential stress is not zero at the circumference, so it will be seen that throughout the plate the tangential stress is larger than the radial stress. It seems obvious that the sum of the two forces, one initial and the other caused by the load, are combined algebraically and that failure occurs at a comparatively low external load.

The cover with concentric ribs should have initial stresses on radial lines between the ribs, Fig. 8(*b*). This is apparently true, for covers made from the same heat withstood a 50% greater load than the cover with radial ribs. All test conditions were equal and the difference in weight was less than 5 per cent. This would bear out the fact that this combination of stresses is not to be neglected if the maximum strength is to be secured.

The contraction stresses should not be of such a serious nature in a cover with a rectangular system of ribs. This is particularly true if the ribs are deep, placing most of the flat cover in compression. These ribs should cause the stress to distribute in a manner similar to the stress distribution in a square plate, Fig. 8(*c*). The contraction of the metal at the intersection of the ribs apparently causes relatively high initial stresses and is the one weak point in this type of design. The greatest strength, however, is obtained with this design.

Summary of Test Results

The results of the test indicate that the two designs for the new standards are satisfactory. The ultimate strengths of these covers are in excess of the design load. For practical considerations the weight of the ventilating cover was increased from 288 lb. to 348 lb.; the distribution of the metal, however, would not increase the strength. The weight of the solid cover was reduced from 419 lb. to 340 lb., because a strength of practically 100 000 lb. is not necessary and the Company did not want a cover that weighed in excess of 350 lb. Table 4 shows the comparison of the ultimate strengths of the covers.

TABLE 4.—Summary of Tests.

Test number.	Record number.	Material.	Weight, in pounds.	Deflection, in inches.	Ultimate load, in pounds.
3	31 086	Cast iron	368	.....	32 400
10	"	" "	371	.....	30 170
2	"	"Semi-steel"	315	.....	27 630
6	"	" "	320	0.25	26 360
7	"	" "	329	0.25	26 360
1	"B"*	" "	338	.....	40 350
4	"	" "	330	.....	37 170
5	"	" "	335	0.22	43 530
14	"	" "	335	.....	40 350
15	"	" "	335	0.27	40 350
8	36 863	" "	419	0.14	68 980
9	"	" "	416	0.244	100 800
12	"	" "	419	0.118	58 400 (Not tested
17	"	" "	419	.....	96 000 to failure)
13	31 086	Cast steel	365	1.310	60 300
16	"	" "	...	0.540 (approx.)	60 000
11	25 370	" "	288	.....	66 750 (Yield point)
A	"	" "	288	.....	100 800

* Record "B", semi-steel cover with concentric circular ribs (Fig. 5).

Conclusions

Table 5 indicates the relative strengths of a single design cast from different metals. Little advantage can be shown for one type of casting over another.

TABLE 5.—COMPARISON OF COSTS FOR DIFFERENT MATERIALS

Type of design.	Metal.	Ultimate strength / Cost
Old Standard	Cast iron	17.0
" "	"Semi-steel"	16.7
" "	Cast steel	18.3

Table 6 shows the relative merits of the different designs.

TABLE 6.—COMPARISON OF COSTS FOR DIFFERENT DESIGNS

Type of design.	Metal.	Ultimate strength / Cost
Old Standard (radial ribs)	"Semi-steel"	16.7
Revised design (concentric ribs)	" "	24.1
New Standard	" "	42.3

A most interesting fact brought out by the comparison in Table 6 is that a 25% increase in weight increased the strength 250 per cent.

The summary, Table 7, shows the saving in cost per installation obtained by this study.

TABLE 7.—COMPARISON OF TOTAL COSTS

Covers.	Cost of Covers. Old.	New.
30-Inch Solid Covers:		
Frames	690-lb. at $0.055 = $38.00	445-lb. at $0.055 = $24.50
Cover (outer)	320-lb. at 0.055 = 17.60	340-lb. at 0.055 = 18.70
Cover (inner)	200-lb. at 0.055 = 11.00	84-lb. at 0.10 = 8.40
	$66.60	$51.60
36-Inch Ventilating Cover:		
Frame	690-lb. at $0.055 = $38.00	445-lb. at $0.055 = $24.50
Cover (outer)	630-lb. at 0.085 = 53.50	348-lb. at 0.085 = 29.60
	$91.50	$54.10
28-Inch Solid Cover:		
Frame	525-lb. at $0.055 = $28.85	360-lb. at $0.055 = $19.80
Cover (outer)	240-lb. at 0.055 = 13.20	226-lb. at 0.055 = 12.40
Cover (inner)	115-lb. at 0.055 = 6.35	54-lb. at 0.10 = 5.40
	$48.40	$37.60
28-Inch Ventilating Cover:		
Frame	525-lb. at $0.055 = $28.85	360-lb. at $0.055 = $19.80
Cover (outer)	390-lb. at 0.085 = 33.20	226-lb. at 0.085 = 19.20
	$62.05	$39.00

The saving in cost for each installation ranges from 22.3 to 40.8 per cent. A considerable part of this was obtained by reducing the weight of the frames as they were exceedingly large, considering the actual load they transmit to

the concrete of the manhole. City ordinances for paving require a certain height for the frame, otherwise the new frame would only be about one-half the height of the old frame. This would be possible as the new inner cover is 0.25-in. plate-dished to conform with the ribs of the cover.

The product "semi-steel" is ideal for the solid manhole covers because the cost is the same as for cast iron and the tensile strength is 50% greater than that for cast iron when approximately 25% of steel scrap is included in the melt. The new covers have been in service for more than two years and no failure has been reported.

Acknowledgments

Report No. 402 by the Bureau of Tests and Inspection of the Pacific Gas and Electric Company contains all the data taken during the test. The Bureau was represented by Mr. A. D. Macintyre, who assisted in making the tests and collecting the data.

R. R. Cowles, Assistant Engineer in the Division of Electric and Steam Distribution, assisted in making arrangements for the test and collecting the necessary data in order to make this study on the covers.

G. H. Delarenelle, Assistant Superintendent of the United Iron Works, assisted in every possible way. The handling of the covers and the operation of the machine was performed by his employees. Equipment for the test was loaned by H. B. Hammill, Assoc. M. Am. Soc. C. E., and by the Pelton Water Wheel Company. The test laboratory of the University of California was used for making the tensile test to determine the properties of the metals.

NORTH CAROLINA BITUMINOUS EARTH ROADS*

By William B. Catchings,† Assoc. M. Am. Soc. C. E.

Soil roads in North Carolina include sand clay, top-soil, shale, chert, marl, spar, etc. Bituminous treatment of these roads gives results in appearance closely resembling sheet or sand asphalt, with practically no failures. During 1925 approximately 400 miles were completed.

Before attempting to treat a road, it is essential that the sub-grade and surface be put in condition to withstand the traffic it is expected to carry. The finished surface will ride no smoother nor will it carry any heavier vehicles than the base provides; therefore, every precaution should be taken in strengthening the base and all possible care used in putting the surface in as smooth a condition as possible. The entire road should be gone over and all weak places repaired with good soil or gravel. Skin-patching alone is not sufficient, as in nine cases out of ten the patch does not bond with the base and later scales under traffic, causing a failure in the surface. All curves of 900-ft. radius or less should be banked.

After the base has been patched, reinforced, and built up, and the curves have been banked, the entire surface should be scarified, machined, and kept dragged, retaining a crown on tangents of from $\frac{1}{4}$ to $\frac{1}{2}$ in. per ft. until the base is thoroughly compacted under traffic. Longitudinal dragging from one side to the other rather than from both sides to the center is recommended. A sectional drag that will follow the contour of the surface is better than one that cuts the quarters only. This preparation is best accomplished in the spring.

Sand for surfacing is kept in stock piles, $\frac{1}{2}$ mile to 1 mile apart, during the winter, to avoid delays during the construction season. Before applying it this sand is hauled out and piled along the side of the road, and "cut back" in a windrow. This allows it to dry quickly and places it in a convenient place for spreading.

Prior to the application of the first coat of bitumen, all loose material is removed from the road surface, first by a road machine and then by a rotary street sweeper. A specially prepared tar is then applied at the rate of 0.3 or 0.4 gal. per sq. yd. and covered with sand to the extent of 12 to 15 lb. per sq. yd. After about six hours this surface is dragged with a sectional broom made of steel bristle push-brooms. This distributes the sand uniformly giving a smooth surface. The prime or first coat is allowed to set under traffic for from 5 to 10 days, in order to allow the volatile oils to evaporate.

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* Presented at the meeting of the Highway Division, New York, N. Y., January 21, 1926. It is expected that the paper by W. W. Crosby, M. Am. Soc. C. E., entitled, "Economic and Engineering Problems of Highway Location", presented at this meeting, will be published in a subsequent number of *Proceedings*.

† Constr. Engr., North Carolina State Highway Comm., Raleigh, N. C.

All loose sand is then swept to the side and 0.4 to 0.5 gal. of a specially prepared asphalt is applied and covered with 30 to 35 lb. of sand per sq. yd. After 4 or 5 hours this coat is also dragged with the broom, which continues with additions of more sand until the asphalt hardens and will hold no more sand.

During 1924 and 1925 asphaltic oils were tried ranging from 55 to 65% asphalt (100 penetration); some straight penetration asphalts and several tars were also used with varying results. From these tests the present materials and methods were determined.

RECENT DEVELOPMENTS IN CONCRETE PAVEMENTS*

By H. Eltinge Breed,† M. Am. Soc. C. E.

The building of concrete roads seems to have entered upon a stage of development comparable to organic evolution; the whole is being modified to an extent that cannot now be foreseen, by changes here and there in detail, often apparently unimportant, often tentative. The main advance is an increasing definiteness about the limits of scientific knowledge. Engineers perceive more clearly than they used to, the hither boundary of what they do not know, and are scientists enough to try to keep pushing that boundary farther off, by investigation, research, experiment, even by trial and error in practice. In other words, while all the factors entering into a problem of highway design are variable, the tendency is toward a mathematical, rather than an empirical, solution of it by trying to fix the limits between which the variables may work.

For instance, by practical tests and the application of the theory of design, there has been evolved a safe formula for the design of corners. Through laboratory research the essentials for obtaining more uniform concrete, such as quality of ingredients and methods of mix, have been brought within greater control. Through soil analysis and research, engineers are learning more about the supporting power of soils. Definite facts have also been obtained through tests, research, and observation, of which the following are examples: "Crack reduction is more economically accomplished by the use of steel reinforcement than by additional thickness of concrete"; and "a greater reduction was afforded by small steel members closely spaced than by larger members widely spaced".‡

Such are a few of the results available from undertakings of recent years. How much their influence on present practice will form the basis for further research is one of the guesses that add zest to life—and engineering.

Confusion may be avoided by discriminating sharply between the design of a rural road and that of a city street. The most important difference is in the bearing power of the subsoil on which the pavement rests.

Before the pavement for a rural highway can be designed, there must be a drainage system to take care of surface water by diversion ditches, slopes, side ditches, and culverts; and to take care of sub-surface water by cut-off drains and outlets. Such drainage must be amplified where necessary by the building of a suitable foundation with proper outlets, by the water-proofing of shoulders, and by the providing of drains along the pave-

* Presented at the meeting of the Highway Division, New York, N. Y., January 21, 1926.
† Cons. Highway Engr., New York, N. Y.
‡ Report, Highway Research Board, National Research Council, Vol. II, 1926.

ment edges. It is most important to prevent the surface water that runs sideways in both directions from the road from concentrating along the edges where it weakens the supporting power of the sub-grade material or adds to its expansive force and to the consequent stress in the pavement slab.

On the other hand, the engineer who designs a city or village street usually has an umbrella over his whole subsoil surface composed of roofs, walks, curbs, gutters, and the pavement itself, to keep the surface water away from the sub-grade. Sewer, water-gas, and other corporation trenches also act as cut-offs for sub-surface water. Furthermore, the pavement usually has a sub-grade compacted by years of usage, often consisting of material with marked stability. Some of the developments to be mentioned are equally applicable to rural and urban works; a few are more important for rural work; and all need to be mixed with shrewd judgment of foundation conditions.

Cracks in concrete pavements have been so costly that many methods have been tried to eliminate or decrease them. Through the use of longitudinal division of the slab, of steel reinforcement, and of limited slab length, the desired result has been accomplished, provided, of course, that the other factors essential to the construction of a good pavement are of such design and under such control as to give a uniform result.

Evidence shows that a slab length between 40 and 60 ft. for rural work, with from 30 to 65 lb. of steel per 100 sq. ft., gives excellent results in freedom from transverse cracks.

Joints as a means of decreasing cracks sometimes seem objectionable because of poor work often done at those points, of weakness that develops unless they are doweled together, and of delay to construction. To offset these disadvantages the State of North Carolina has built a test section of pavement in which planes of weakness are introduced every 40 ft., so that the slab will crack and divide itself, thus dispensing with joints. The steel reinforcement is omitted for a width of 1 ft. transversely across the pavement. After "belting", a battered wood strip, of ½-in. top, and ¼-in. bottom, width, and 2-in. depth, is forced into the concrete flush with the surface where the steel has been omitted. The surface at the strip is finished smooth with a trowel. After the initial set the board is pulled out, leaving a slot in the concrete which is filled with tar after the curing.

This method permits the speeding up of construction. It gives an interlock at the joint which will act as long as it is closely held together. Without steel, however, it tends to be forced farther and farther apart as time goes on. The main disadvantage of this method is that it does not provide for subsequent expansion, which, according to many authorities, is cumulative. If room for expansion is not provided in a longitudinal direction, ultimately there will be crushing of the concrete at the cracks, or diagonal cracking throughout the slab, where the accumulated stress from elongation is greater than the weakest parts of the slab. This work in North Carolina was constructed during March, 1925. During the curing period a crack formed, as was expected, at each plane of weakness.

An inspection in October, 1925, on a cool, cloudy day, showed open cracks at all the planes of weakness and no other transverse cracks in the 2 000 lin. ft. of pavement under the test. This is the only method yet devised of securing regular breaks in a pavement in which it is desired to omit the regular joints during construction.

Another recent development is the two-course pavement, which is again coming into general use as an outgrowth of sidewalk construction. This type has been proved to be a success by the wonderful service given by many of the old Blome concrete pavements. It has been used extensively, especially in the environs of Buffalo, N. Y., where there is a scarcity of good materials for concrete roads and pavements except at large expense for importation. Since 1922 more than 1 000 000 sq. yd. of this type has been constructed as follows: Total thickness, 8 in., the bottom 6 in. being of 1:2:4 concrete with gravel, stone, or slag aggregates; while the top 2 in. is of 1:1.5:2 concrete, using gravel up to $\frac{5}{8}$ in. in size; all other features were those usual to good construction. In some localities where good aggregates for the top are not available, a mortar top has been used with excellent results. Economically this type has the advantage of low cost on account of making poorer materials available for use. The fine aggregate or mortar top decreases materially the cost of surface finishing, and the richer mixture in the top surface adds materially to the resistance to abrasion, as has been shown by a number of investigators. This resistance has come to be a considerable factor where tire chains concentrate wear for many days at a time in ruts made through the snow.

Chapel Street, New Haven, Conn., is an excellent example of this type. This pavement, built in 1908, was of the Blome Granitoid type, more than 3 000 ft. long, on a main thoroughfare from the east, and probably the heaviest traveled street in the city. The pavement is $6\frac{3}{4}$ in. thick, the lower 5 in. being a 1:3:4 concrete, covered by a $1\frac{3}{4}$ in. top, composed of 1 part cement to $1\frac{1}{2}$ parts of clean crushed trap-rock having a maximum size of $\frac{1}{4}$ in. and the dust removed. The original markings are still in evidence after seventeen years, and the pavement is entirely serviceable, which is remarkable considering the number of steel-tired coal trucks that have passed over it.

Another illustration is a rural highway built under the writer's direction at Finley Lake, N. Y. This pavement was 4.33 miles long, with a 28-ft. roadway; the concrete surfacing, 10 ft. wide, and 6 in. thick, has a 4-in. base of 1:2:4 gravel concrete and a 2-in top of 1:2:3 imported stone. After seven years it is still in excellent condition.

An interesting invention has been made by E. G. Hooper, Assoc. M. Am. Soc. C. E., to meet the difficulties of uncertain conditions in drainage and subsoil. Where these factors are indeterminate, the logical plan is to eliminate the roadbed as a supporting medium, and predicate the design on artificially established supports for the pavement. Thus, a 20-foot pavement is made up of two 10-ft. lanes of equilateral triangular slabs, each slab being supported by a pile at each of its three apices (corners). Each support carries the end of one or more adjacent units. Adjacent edges are designed to inter-

lock to make adjoining slabs mutually supporting so that settlement of any corner will cause no appreciable change in design stresses. As the supports are carried to a solid foundation and below frost, they may be considered to be permanent. The moments and shears for the given loads may be calculated. Confirmation of the validity of the method of design has already been obtained through the use of test models. This type of pavement offers a promising solution for viaducts or for heavy fill sections over unstable soil.

On the theory of design based on the tests mentioned, a pavement thickness of 7 in. with 27 lb. of steel per sq. yd. is found adequate to carry the 12-ton axle load often specified for a 15-ton loaded truck. The cost of the pavement proper per unit of material used is not greater than for the usual pavement.

Among the most important of all recent developments is the application to practice of the researches* of Duff A. Abrams, M. Am. Soc. C. E. These researches definitely show that an increase in the water-ratio (wetter concrete), a decrease of cement, an excess of fine aggregate, or a poor grading, mixing, or curing, diminish the compressive and bond strengths of the concrete, and lessen its impermeability and its resistance to wear and destructive agencies. Had the results of this work been available ten years ago there would have been saved large sums invested in concrete that is already showing signs of limited life.

Every lad knows that he can make synthetic stone by mixing sand, gravel, cement, and water, and some concrete structures built with no more skill have miraculously survived. However, with a steadily increasing public demand that all sorts of structures beside pavements shall be of concrete, it behooves engineers to safeguard public funds by improving the knowledge of the scientific details of concrete work to the limit of their ability. Recently, $2 000 was wasted in one small town because a shoulder extension to a concrete pavement was laid flush to the curb without a particle of allowance for expansion; and the engineer in charge of the job was blissfully complacent!

Concrete of high early strength is coming more and more into demand for streets under heavy traffic, for emergency work, for replacements, bridge floors, cold-weather work, and, in general, wherever it is necessary to shorten the period of curing. Properly controlled, 1:2.5:4.5 concrete, with 1.32 bbl. of cement per cu. yd. and 7.8 gal. of water per sack, will give a compressive strength of 750 lb. in 3 days. Changing the mix to 1:1.5:2.5, the quantity of water to 4.4 gal. per sack, and increasing the time of mixing to 5 min. will give a concrete of a compressive strength of 2 860 lb. in 3 days, which is almost four times as strong as the usual mix at that age. If still further acceleration is required, or if only the usual mix can be used, high early strength may be attained by the admixture of calcium chloride. This recent development is coming into general use and giving satisfaction. It should be safeguarded, however, by observing in a preliminary test to just what extent the calcium chloride hastens the set of the particular brand of cement to be used.

Important for safety is the increasing tendency to straighten alignments and build wider pavements. Engineers have advocated this for years; people

* *Bulletins Nos. 1–17*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

are just beginning to demand it, and every effort should be made to keep before them the necessity for the jump from the 20 to the 40-ft. pavement in congested areas. The 30-ft. or 3-lane width on busy thoroughfares is a snare, especially if the traffic is evenly distributed both ways. That middle strip invites two cars to turn out at once and come head on to each other. With 20 000 000 cars now in use in the United States, and 3 000 000 more being built this year (1926), the public must realize, and engineers must help make them realize, that adequate roads mean not only safeguarding life, but also actual money saving in time, gasoline, and nervous energy.

Such are some of the important recent developments in concrete road building. Most important, perhaps, is the progress in knowledge and education in regard to all types of highway construction. For their large part in this progress tribute should be paid to those organizations which have made unfailing efforts to attain and maintain standards: The U. S. Bureau of Public Roads, the Asphalt Association, the Portland Cement Association, the Brick Association, the various engineering societies, and the American Society for Testing Materials. Upon them, and upon the engineers on the jobs, depends the success of American roads.

QUALIFYING ENGINEERS FOR HIGH EXECUTIVE POSITIONS

AN INFORMAL DISCUSSION*

By Messrs. E. M. Herr, H. A. Guess, F. B. Jewett, and J. C. Parker.

E. M. Herr,† Esq.—That executives in important positions should have engineering training has long been recognized in many American industries, as well as in some railroads, which have made it a point to select their chief executives only from men with such training.

What are the requirements engineers must have to qualify for these high executive positions? They must be men with ability to lead and direct others; they must be men of decision and initiative, trained in evaluating men and materials; they must be men of vision and of courage—not reckless, but with full appreciation of their responsibility for the results of their acts and decisions; and perhaps, most of all, they must be men able to make proper decisions as to economic conditions and prospects. Executive ability is the ability to guide and direct others. It is inborn, and not acquired. Where it exists, it can be developed and strengthened by practice and training.

How can men be qualified for executive positions? In the first place, great care must be exercised, in outlining a course of study for young men looking toward their occupying executive positions, not to give them the idea that a course of study alone can fit them for such positions any more than a course in music can insure that the pupil will become a virtuoso or operatic star. The ability to reach the higher positions in any field must reside in the individual himself. This does not cause hesitation in educating young men for professions, nor should it do so in training them for executive work.

Even though one is gifted with the qualities of leadership, it is only by persistent, painstaking effort to make good in the ordinary or work-a-day requirements, and to gain a thorough understanding of the details of the business, that an opportunity is finally obtained to occupy the higher executive positions.

An engineer with executive work in industry in view should first be thoroughly trained in the underlying fundamental engineering studies, so that he will not only understand them, but will have mastered them in such a way that he can use them readily and accurately in any investigations it is necessary for him to make. Having this foundation, he can then take up the subjects that enter into the administration of an engineering industry

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† Pres., Westinghouse Elec. & Mfg. Co., New York, N. Y.

and familiarize himself with the principles underlying them so that he will understand how they enter into the business under consideration. He should also take such humanistic and cultural studies as will give him a well-rounded and thorough intellectual training. In this should be included a good grounding in the English language so that he will be able to express himself clearly and readily, either in writing or in speaking.

To acquire even a moderate knowledge of the principles underlying the the different departments of any important industry will require all the available time of the student, and will necessitate that he devote less time to the purely engineering studies leading to inventive or design work. He should use practically all the long vacation periods of the college year to acquire knowledge of those things which can only be gained in the shop or in the field. By following such a course with diligence, the student should acquire training that will enable him to work into an executive position, if he has it in him; and how high he will rise in executive work will depend on how well he can qualify.

Nearly all industries must now develop an export department having contact with foreign countries. A knowledge of one or more modern languages is, therefore, important and should be included in the course of training.

The technical schools and colleges should recognize the value of the extra curriculum activities in developing leadership and executive faculties in the students and make a vigorous effort to include in the regular curriculum activities that will bring out the same faculties. If successful, such a move would make training in leadership and management a part of the regular college work.

It must be clearly kept in mind that no amount of study and no amount of training will enable a young man to become a high executive, or even to occupy an executive position of any kind unless he has the ability to direct others, and, by reason of his knowledge of the business, to take the lead in the work to be done. He may be a remarkable designer, an inventor of high attainments, a skilled technician, or he may have wonderful qualifications in many directions, but unless he can lead and direct men, he cannot qualify as an executive. Nor can every leader of men qualify as a high executive, or be placed in charge of a great industry. Here, the knowledge of business conditions, the quality of leadership secured only after years of business training, the business judgment, the courage, persistency, and will power to carry through, all of which can only be obtained in the hard school of actual experience, are vitally needed in the attainment of a high executive position. Manifestly, these cannot be learned in any school or college, but must be inborn in the man himself, awaiting development by environment and opportunity in the business undertaken.

Given all these things, there is yet another quality demanded of one who would direct a great industry—the ability to forget one's own personal wishes and plans and be willing to make any sacrifice, to place one's whole self except one's honor upon the altar of business achievement.

In former times an executive in a high or commanding position in an industry was either the principal owner, or had back of him an individual or a definite group of owners who controlled the business. To-day the trend is toward such diversification of ownership that no one individual or responsible group of owners really controls. This places squarely upon the board of directors and executive officers the responsibility for the conduct of these large industries. There being no large part of the business owned by any interest that would be justified in devoting sufficient time and thought to its future to be of any real assistance in guiding its destiny, the chief executives must, of necessity, be men not only of ability but of the highest integrity and character.

Americans are generally so much concerned with business and its material results that there is grave danger that young people will be trained too much along material lines, and not enough along human and spiritual lines. Thus far, the speaker has dwelt only on education in the material and physical side of the business. There is also a human and personal side in every undertaking, and educational courses will be badly misdirected and one-sided unless those who are destined to guide the industries are also trained and developed along this side as well.

The building of character must not only be included in the education of the youth of the nation, but this must continue throughout the business career of those who hold high executive positions; in fact, one of the most important as well as one of the most difficult duties of a chief executive is that of providing for the personal development of those who comprise the management—those executives who administer the business. If they are men of high character, such will be the character of those whom they employ and direct; and the character of those in responsible charge of an industry is always reflected in the quality and reputation of its product, and in the treatment of its patrons.

From this brief outline, it is evident that executives for the higher positions cannot be produced by college training and education, nor can they be produced at all unless the man has in him the real quality of leadership. If a man has this quality, it can be stimulated and developed by proper education and training, but this training must be largely in the business itself.

His engineering education must be to a great extent in fundamentals, but must be most thorough. His general education must be broad enough to include those studies which deal with the humanities, as well as cultural studies embracing one or two modern languages. Extra curriculum work should be encouraged, especially where the opportunity for leadership exists.

Finally, in order to attain the highest positions, unselfish, self-sacrificing devotion to work must prevail to the highest degree.

H. A. Guess,* Esq.—If the speaker can say anything of interest or value upon the subject of "Qualifying Engineers for High Executive Positions", it can only be in relation to personal experiences and observations spread

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over a period of thirty years as a continuously active mining and metallurgical engineer. All these remarks, therefore, are to be understood without further qualifying as referring to mining and metallurgical engineers.

At the outset while the Speaker believes that engineering courses in the various colleges and universities could be arranged to give the student within the same time limit a broader general education of undoubted value, still he does not agree with the view held by many that engineers in comparison with those of other occupations—for example, doctors, lawyers, bankers, and so-called business men—seldom arrive at high executive positions.

After all engineers are not very numerous as compared with these other occupations and, therefore, a lesser number reaching high executive positions, stated numerically, might be a larger number, stated as a percentage.

Pursuing this matter of comparisons and considering now, doctors: Every one knows there must be throughout the United States, say, three doctors to one engineer; but even numerically probably there are not as many doctors in high executive positions as engineers; and it is a fortunate thing for the country that doctors as a class are so busily engaged in alleviating sickness and suffering and in the advancement of medical science, that they do not aim particularly for high executive positions.

Consider next the legal profession. Without looking up a census of all the lawyers in the United States, doubtless there are at least two lawyers to one engineer. Further, it must not be forgotten that the ever-increasing complexity, during the past twenty years, of laws with respect to personal and corporate behavior, and the penalties for inadvertent or other misbehavior either of omission or commission have been a veritable manna from Heaven to the lawyers, to whom, with growing frequency, every individual and corporation must turn for legal guidance.

Under these circumstances it seems little short of a miracle, that to-day there are any high executive positions not occupied by lawyers. However, as a matter of fact, while a number of the large corporations have as their head or sub-head a seasoned lawyer of wide business experience, it is doubtful if there are in high executive positions to-day, as many lawyers as engineers, stated as percentages of the total number of each profession.

The two remaining classes of the occupations previously mentioned for comparison, are bankers and so-called business men, which would include manufacturers, merchants, etc. Lumping these together as one gives a class much more numerous than the combined doctors, lawyers, and engineers.

It is natural, therefore, that from this class should come those holding the majority of the high executive positions, because even though not engineers, they are very clear headed, are in many cases college graduates, and have been trained in the greatest of all universities, namely, the modern business world.

On the whole, therefore, the Engineering Profession is securing about its percentage share of the high executive positions in America, with the possible exception of the banker class. The banker, however, is the custodian of cash, of real "high-grade ore" to use a miner's expression, and every one knows

how important and how looked up to, even among engineers, is the head of a big high-grade mine. He looms up like an Oregon fir among quaking asp.

There remains then to consider in what way during his college term and after, can the engineer be trained still better to qualify for high executive positions. The answer is not simple, because fortunately for the joy of living there is an endless variation among individuals, and there is no known method by which certain ingredients can be added to, and taken from, this individual, and similar needful ministrations given to the next, until all are standardized—a 100% finished product.

However, most old graduates whose opportunities for observation have been good, are of the opinion that a large amount of instruction on detailed mining and metallurgical methods could be dropped because much of it is obsolete, at least by the time the student is graduated, and much also is of a sort that can be understood better by a few days' visit to an operating plant than by weeks of classroom or laboratory study.

The time thus made available could be devoted to the student along lines, say, as follows:

First.—Giving a sound acquaintance with modern business systems of accounting with more particular reference to segregation of costs in mining and allied operations, so that the cost of each operation can be determined and scrutinized and then possibly reduced. Such study in addition to giving the student a much needed acquaintance with modern accounting methods would inculcate the habit of considering any mining enterprise from the viewpoint of current and ultimate profit or loss.

Second.—Stressing the basic importance of the human element in all operations and the vital necessity, therefore, of being not only a thorough student of human nature, but a sympathetic one, so that the young graduate will instinctively be a good judge of men and, at the same time, earn their respect and good-will instead of finding it difficult.

Third.—Arranging that the student shall become accustomed to expressing himself clearly and succinctly both in writing and verbally. Most of the better engineering schools include now as their requirement a sufficient study of English composition and literature to ensure with average mentality a fair degree of clarity in writing, but ease and clarity in speaking are of nearly equal importance, and should if possible be instilled not necessarily or even preferably as a course in rhetoric or public speaking, but as an informal, open-meeting method, either conducted directly by the professor, or indirectly through an organization within, say, the senior student class in engineering.

By omitting a considerable amount of instruction on mining and metallurgical detailed methods as previously suggested, it would also be possible to strengthen the instruction on the fundamental principles of engineering so that the graduate would be better equipped to meet intelligently diversified problems when they arise, of a sort that necessarily no college instruction could anticipate and cover as to detailed solution. At the same time it must always be remembered that the engineering course should give enough detailed instruction and practice in two or preferably three branches, so

that the young graduate may not encounter too much difficulty or delay in finding a job, because, unless he is financially independent and also has qualities quite above the average, the only way to start an engineering career is as an employee.

Nowadays, many of the larger mining organizations employ a certain number of fresh graduates each year as assistants on underground sampling crews, or as workmen around a mill or other metallurgical plant, or as juniors in a field examination force. From this work the engineer in course of time progresses to a better position according to his abilities and the chance of circumstances.

However, the better positions available each year represent at best only a small number compared to the total yearly graduates, and in general, therefore, the young graduate must look for employment to the various mining and metallurgical companies mostly of small or medium size, that may need certain skilled services. For this reason mining schools should give sufficiently detailed instruction and practice in surveying, drafting, and assaying to enable the average graduate at least to hold such a job when once obtained.

The young graduate suitably qualified, on obtaining such a position, will naturally become quite proficient in it and will find time to acquaint himself partly with various other activities around the mine or plant, and in due course of time will doubtless have opportunities for more important service.

The foregoing suggestions are offered with considerable diffidence because bright minds have been working on this subject of engineering courses for many years, and also because the present engineering courses, judging from the quality of the product turned out year by year, appear to be functioning fairly well anyhow, considering their widely assorted raw material.

After graduating and securing a job as a start, the engineer's progress other than as modified by fortune or misfortune depends almost entirely on himself. His progress, however, will doubtless be assisted if early in his career he is fortunate enough to be employed by people whose enterprises are managed with skill and energy, as he will thereby derive both stimulus to high endeavor, and a great fund of knowledge along proper lines.

Probably all will agree that the qualities that are or should be characteristic of high executives are somewhat as follows: Integrity, sustained energy, skilled knowledge, tact, personality, and basic common sense. Certainly not many of these can be taught as subjects in a college course, nor can they be taught later; at best, they can only be stimulated and fostered.

However, the picture would scarcely be complete if one refrained from stating the obvious truth, sometimes unfortunately lost sight of by the one chiefly concerned, that no matter how high the executive or how unusual and admirable his qualities, the career that he has apparently carved out for himself is due in no small degree to his competent assistants and associates and also to the favors of fortune, the "rub of the green" to use a golfing expression. Shakespeare doubtless had this in mind when he had Hamlet say:

> "There's a divinity that shapes our ends,
> Rough-hew them how we will."

Others to whom in spite of their earnest and intelligent endeavors, fortune has been less kind, are at times inclined to believe that this quotation should have been punctuated differently and should have read:

> "There's a divinity that shapes our ends rough—
> "Hew them how we will."

F. B. JEWETT,* ESQ.—When one considers the vast variety of ways in which successful business operations and organizations have been built up, and the even greater variety of characteristics embodied in their executive officers, one is compelled to broach the making of categorical statements relative to these qualifications with a great deal of trepidation. Obviously, no simple set of cardinal qualifications nor any simple rules for the development of desired characteristics are universally applicable. As no two business organizations are ever exactly alike, there is evidently the widest latitude for the exercise of individual ability and talent in the rise to power and influence of the executive.

Every one who is engaged in work that involves so-called executive responsibility, will have necessarily a somewhat special view of the characteristics which he considers essential in the make-up of a good executive. This point of view is largely an outgrowth of his own accumulated experience and observation as seen through his own eyes. No one need be surprised therefore if the points of view of the several discussions are divergent as to the relative importance of different factors. They may be so divergent, in fact, that matters which appear of importance to one may not even be mentioned by others.

Nevertheless since presumably this discussion is dealing with the qualifications of executive officers in responsible charge of organizations that have more than a transitory existence, there are certain general qualifications which, all will agree, are requisite to success in any field. These are the qualifications which human experience through untold ages has demonstrated as essential in the make-up of men who would lead other men and direct their joint activity in the furtherance of a common objective.

First among these of course is character—a thing which comes into the world with the human being himself. While training, example, and environment may possibly modify character to a degree, its essential quality is a heritage from one's more or less remote ancestry.

Second is the quality called "personality". Since personality in the successful executive involves the element of character it is not easy to define just what meaning is intended. What is in mind unquestionably is the peculiarity possessed by some men which impels their fellows instinctively to seek respect and follow their leadership. Any natural inclination of the individual to lead is fostered by this spontaneous attitude of his fellows.

The thing the speaker has in mind when he thinks of personality in the successful executive is not any quality of the moment, nor is it a quality with which the individual can consciously invest himself. Like character, it is a thing of which he is born possessed and with which he goes through life.

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Third among the general characteristics of the successful executive is the attitude of mind with which he encounters all the problems which present themselves to him. It is an attitude, compounded of imagination and initiative, of a broad understanding of and sympathy with human values, of experience which leads him to temper the claims of the enthusiast and the reactions of the stand-pat conservative—added to which is the will and determination to adventure forth on whatever course his mature judgment determines to be correct.

To consider engineers in connection with high executive positions is to consider them first, of course, in connection with those activities that are in some way involved in engineering. In its present-day civil sense engineering is a relatively new factor in business. It is by far the most important single factor that has been introduced in modern times. This is so both because so much of modern business is inextricably involved in engineering, and also because the other great factor—size—which appertains to present-day business, is itself the direct or indirect result of the engineering factor.

With the element of engineering so prominently involved in modern business, and with the prospect of its increasing importance as pure and applied science unfolds, it is clear that the successful executives of the future must come in large measure from those men who, in addition to their other qualifications, are well versed in the principles of engineering and the sciences on which engineering rests.

Doubtless many successful executives directing great engineering industries will come from the ranks of those who have had little or no training or experience in the purely technical field. Such men, however, will be the exception rather than the rule. As modern existence becomes more and more enmeshed with the things that science and engineering produce, the proper operation of these things in their relation to life and their orderly conduct for the good, rather than the hurt, of society, inevitably, it seems to the speaker, demands the leadership of men who are versed in science and engineering. No other group of men can aspire to that understanding of underlying factors on which, in the last analysis, the whole structure rests.

If this concept of future requirements is correct it behooves industrial leaders to seek out and, at the earliest possible age, to develop, and to elevate to positions of authority, those engineers who possess the necessary qualifications to act successfully in high executive positions.

In many ways the growth of industry based on engineering during the past three or four decades has been distinctly opposite to the direction which would lead the engineer to a high executive position. In part, this tendency has been the result of functional specialization, and, in part, the result of the great inherent interest in engineering problems *per se*. Many engineers otherwise qualified for general administrative positions have led lives of profitable enjoyment in the field of their purely technical activities. Frequently, this adherence to their specialized field has been of great advantage to their industry and the community. Sometimes, the speaker ventures to believe, it has cramped the fullest development of the industry by depriving it of the best leadership.

Since some of the cardinal characteristics of the successful high executive are human attributes not involved in the field of chosen activity, it is clear that only a few of those who embark on engineering can hope to achieve as executives. The speaker's experience, however, leads him to believe that the proportion of men with executive qualities who embark on a career of engineering is, if anything, greater than the proportion in many other walks of life. The training and early experience demanded of those who would enter engineering are such that only those from the upper strata of intellectual capacity can venture into it. If, therefore, it may be assumed, as doubtless it may, that there are in the ranks of each generation of engineers men with every inherent qualification for high executive positions, the problem is simplified and becomes merely one of selection and conscious development of the individual.

The qualities of character, personality, and attitude of mind tend early to evidence themselves. Those who are interested in their successors have only to look, therefore, for the evidences of these qualities. Having found the young engineer who seems to possess innate facility for an executive position, it should be their object to direct his career so as to steer him clear of a too narrow technical specialization, and into fields where in addition to maturing as an engineer he will also gain experience as a leader of men and as one is familiar with the general problems of business.

Doubtless they will make many mistakes, particularly in the early stages of their choice, but without question out of it all will gradually emerge a group of men who at middle life are not only capable engineers but are also experienced in the realm of leadership and the problems of business administration. From this group the final selections for high executive positions can be made with a certainty far removed from the haphazard.

Apparently the speaker has approached the subject from a slightly different point of view from that of Messrs. Herr and Guess, having in mind not so much what the college, the university, the technical school, and the man himself might do, but primarily how to supplement the education of engineers, to provide adequately for the future administration and conduct of business.

A great many years ago, as a minor executive the speaker was confronted with numerous problems looking toward the future. At that time he came to two conclusions, neither of which has he had cause to change in the years that have passed—one was with regard to what was required to make a good engineer; and the other was as to what was required to provide the necessary executives for engineering business.

Unquestionably the first requires in the formal training of a man the most thorough possible grounding in the fundamentals of his profession. In the formal part of his education, there is not so much need of a lot of detail work, such as engineers of former times went through, as there is for emphasis on what really constitutes the fundamentals.

The second conclusion was that in the increasing complexity of business founded on engineering, the executive who himself did not have first-hand knowledge of the engineering features of his problem was at a great disad-

vantage because so many of the decisions affecting business policies are in the last analysis successful or otherwise to the degree in which the decision is based on a correct appraisement of the technical side of the problem.

The correct answer frequently lies not in business judgment of the usual kind, but in the physics of the subject. According to the speaker's observation, even with the best intentions in the world the executive who had to depend for that phase of his judgment on somebody else, and who had no personal judgment in the matter, was at a very great disadvantage.

These thoughts led the speaker to look about in his own domain to see what was being done to provide for the future. After taking stock of the situation, it seemed that his company was doing very little and leaving a great deal to chance. It was taking what came out of the rut of life and was not using its facilities to the best advantage to search out or consciously attract young men toward a life of executive control. This state of affairs led him in the last ten or fifteen years consciously to seek out those young men who seem to have the characteristics of leadership in technical problems and to broaden their experience by shifting them from one type of work to another as long as they show qualities of the leadership that is required. The ultimate aim has been to produce a reservoir of men who at middle life would be qualified both technically and by experience to administer the affairs of a great technical business.

It is still too early to say how successful that conscious effort has been or will be, but to date it looks promising.

J. C. PARKER,* M. AM. SOC. C. E.—An accusation of presumption might easily be raised against one who consents to talk on this subject, particularly for one who, in experience, at least, is not yet quite "dry behind the ears". Therefore, let it be confessed that one speaks "as the Scribes and Pharisees" and not "as one having authority". Certain things which really need to be said are here offered for what they may be worth and not with the idea that any significance attaches to this particular speaker's utterance of them.

It will be necessary to describe personal qualifications and characteristics. Whenever a man does that sort of thing it is pretty likely to be assumed that he is painting his own portrait with all the imperfections and bias that attach to auto-observation. This assumption is not quite correct. Each one must believe that the kind of a man who is qualified for the kind of a job that each aspires to is exactly the kind of a man that each individually tries to be. There can be no criticism of this kind of a process. For a man to try to be other than what he thinks he should be is deliberately to head wrong, and it is doubtful if any human being ever does quite that. By the same token, setting up the picture of what one would be, of what one strives to be, is a long, long way from claiming to have made a decent portrayal of what one has become.

"Qualifying the Engineer for High Executive Position"—what does this imply? Training him to get that kind of a job, or to hold it after he has got it, or to get the maximum for himself out of what he holds? If so, a

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group of representative professional men can be expected to discuss such a matter only on the rather derogatory assumption that they expect out of it to learn some of the "tricks of the game", which "tricks" most probably would be jealously guarded as the private secrets of those who think that they have acquired them. To the credit of the profession be it said that engineers characteristically are more interested in duty and in righteousness than in self. Therefore, it may be assumed that they concern themselves with that form of fitting the engineer for that kind of an executive job which best shall enable him to do his share in advancing the world's progress. Not otherwise can one expect others to take much interest in the judgments of executives and of those who aspire to such dizzy eminences.

It is well that the subject states the problem as one of "qualifying" the engineer. This suggests that not customarily are the executive jobs held by engineers, or at least not so commonly as professionally engineers would wish. This seems to be the case but it is doubtful if the impression would be strengthened by statistical data. Perhaps, rather, when an engineer becomes an executive he ceases to look like an engineer and so perhaps other engineers fail to classify him as attached to the profession. None the less, most engineers do feel that too many executive jobs are filled from professions other than their own even if the enterprises are closely related to engineering.

There must be some fault in themselves if this be so. It is safe to start with the presumption that, in the main, stockholders and directors will fill positions on merit and not as a result of prejudice, whim, and caprice. Sheer self-interest would lead in that direction and although engineers may have failed to attain all that they would desire in the control of industry, they probably have attained all that they deserve.

There are exceptions to this principle. Too often circumstance has placed unworthy individuals in control and kept worthier ones in a position of subordination but it is hard to believe that such an inversion of reason can ever for any protracted period extend to a whole group. If, intrinsically, engineers have merited more authority than they have been able to persuade others to yield to them, this latter fact in itself is a fault in the profession. The purpose of this discussion is to try to find what must be done by the individual engineer in order to overcome any such defects on the assumption that when the qualifications exist, the opportunity to exercise them sooner or later will be forthcoming.

Probably the best way to determine the qualifications of an executive will be to observe those who most successfully have filled executive positions. Frankly, the speaker has not found that the qualifications for an executive job are much different from those that ideally may be required of any other workman, always, of course, differentiating qualification from mere attainment, which does not invariably imply any qualification whatsoever but is in its nature incidental and perhaps may be quite accidental.

Often one hears disparagement of mere technical training as preparation for growth. Is not the stress wrongly applied? Is a man less fitted to become a good boss-carpenter because of skill in handling, and a profound

respect, for his tools? Is not any disqualification rather in the fact that one may become so concerned with technique and so enamored of technical agencies as to lose a sense of their proper relation to life?

It is irritating beyond measure to meet in any calling that narrow-minded infatuation with technique which brushes aside common sense considerations. This, however, is simply not good technique. The engineer who designs ever so good a device which society does not want may be an excellent scientist, even a skillful designer, but he is not an engineer in the sense that engineering is an adaptation of scientific and economic process to the service of mankind.

Is this obsession by the technique of the trade, the science, or the art at which one works any more of a disqualification than is a too complete reliance on the collateral elements in performance? Have we not all seen the boss-carpenter, the physician, the priest, the counselor-at-law, and even the engineer, who becomes so much concerned in business-getting methods, in suavity, in personal advancement, as to forget the solid technical groundwork without which he may, it is true, have become financially prosperous, popular, eminent, or notorious, but none the less, lacking which he scarcely can claim to have attained any of these things as an apostle of his own particular vocation. "These ought ye to have done, and not to leave the other undone."

First, then, among the qualifications of an engineer for an executive position is that he be an especially good engineer—as good an engineer as he knows how to be; that he have his technical feet definitely under him, not relying on pinchbeck substitutes.

Of course, one does not assume that it is particularly important in itself that an engineer know everything about transient phenomena, surges, ellipsoids of stress and entropy, but rather that he have at least a speaking acquaintance with the essentials of engineering science, and, more important still, that the processes of objective observation and of orderly thinking shall have developed as the essence of scientific method.

This does not require that one be familiar with the "lingo" of physical science and that he be able to juggle mathematical symbols, but rather that he have the scientific habit of mind as an intuitive, rather than a conscious, thing. Too often there is an inclination to mistake the machinery for the motive power, and to assume pages of mathematical formulas to be evidences of erudition when perhaps they are essential displays of quite the opposite; exactly as a scrupulous devotion to the book of etiquette marks one with the discriminating as being not quite a gentleman however it may fool the uninitiated. It is assumed that a good technical training and familiarity with scientific methods will result in fundamental simplicity of thought, of mental processes, and of expression, and that this is the measure of real science.

Without doubt a man who in engineering design can differentiate fundamentals from superficialities, essentials from incidental details, and who can, in a simple, straightforward manner, make deductions and arrive at con-

clusions, will be able to approach economic problems and human relations with a reasonable measure of success. This perhaps is something different from being a mere technician.

Perhaps one of the best by-products of the scientific training is the ability to generalize; to proceed from an adequate mass of particular experiences to the formulation of general guiding principles which shall control individual performances and herein is one of the principal differences between the true executive and the "detail hound". The latter concerns himself with the individual experience as a thing sacred in and of itself, and fails, no matter how many such experiences he has had, to draw from them guiding principles which quickly shall illuminate and mould the subsequent course of life. Such a one necessarily "wabbles" when he is confronted with new problems or has recourse to generalities, which are utterly different from generalizations. The process of generalization results in decision, correct at least in its essentials and easily enough modified in the detail of application.

This does not for a moment assume technical analysis to be a good scientific process if it is a cold, hard, sterile thing. The speaker cannot conceive the design of a power plant, the location of a railroad, the development of waterways and harbors as a thing apart from life. Hence, the best engineer will realize that his engineering is dust and ashes unless it concerns itself always with the humane purposes that it is planned to accomplish and out of which arises its only justification.

Engineers at times like to call themselves "sons of Martha" and in that little boast they show that they have entirely missed the whole point of the story which is, as the speaker recalls it, rather definitely that Mary had chosen the better part. Let there be no mistake about this. With all other professional men engineers are definitely hired to render a service to society. For example, a highway improvement is asked for. To what purpose? Well, among other things, in order that people may travel for enjoyment, or get to and from suburban homes, or to their golf links. Now, if the highway engineer is entirely convinced that the shortest distance between two points is a straight line irrespective of the vested rights of a century-old elm tree here, an interesting knoll there, he may produce a finely efficient avenue of transportation and in the process defeat the very purposes of human happiness for which the highway was created.

Or, again, a manufacturing enterprise existing supposedly to produce the comforts of life and the means to prosperity for an American clientele, is so managed as to get out quantity production at the least possible cost. This is fine as far as it goes, but does it not go too far and defeat its own social purposes if in the process vocational break-down and un-American class hatreds are developed by subordinating humanity to the technique of production? The engineer—be he executive or junior employee—besides being technically expert and scientific in his process, must be socially-minded.

A social-mindedness that attempts to embrace humanity in the mass is sure to be a pretty vague thing and liable to a great deal of the academic. A

social-mindedness that arises out of and draws on contact with individuals of all sorts and conditions, the good and the bad, the weak, the strong, the stupid, the brilliant, that can make allowances for them all, that recognizes in each a great deal of the virtues and some of the vices of oneself, is bound fairly accurately to interpret the wishes of the society of individuals which one must serve.

The ideal engineering executive, then, whether dealing with customers, with the general public, or with employees, if truly social in his habits of thought, will put himself into the place of various individuals in the group, will apprehend their reactions to, their interest in, and their rights in respect of, his authoritative decisions, and will lend a human flexibility to what otherwise might be a steel-hard determination.

Nor does the speaker deem the development of human experience to be a thing which one attains, but rather a thing into which continuously he grows. There is no graduation from the school of human conduct but, rather, a continuous commencement from the preparatory into the wider fields. Therefore, the executive after he has been nicely ticketed and labeled as such, will keep alive his training in human and social relations.

Now, no training in human relations, any more than an art, a sport, or a science, can be had as a mere act of will. Back of it there must be an inspiration, something of a divine fire. Nothing in this world that is worthwhile can be sucked dry and thrown away when all the juice has been extracted. That kind of a process simply brings one nowhere, and so a knowledge of men's feelings, and reactions, and impulses, is to be had not by subjecting life to the microscope, but by living with, and in the lives of, other men rather than merely using and observing them. The office boy, the subway guard, and the corner fruit vendor are just about as human as other people, if not more so. They have much to give but they will not allow anything to be taken from them. Every one experiences a similar feeling. An income tax declaration extracts as much as one wishes to yield; to friendship he gives all that he has. This is to suggest somewhat of democracy and kindliness in the least obviously significant of human relations—definitely as training, if no higher motive can be found. However, it is much to be doubted whether democracy and kindliness can be acts of conscious intent. No one particularly cares for consideration that is extended through a sense of Christian duty; nor does he respond to that sort of thing. Rather, it seems, a decent modesty will tell each one that he is made of the common clay, will give each a kindliness toward his fellow unfortunates who need so much in the way of cheer and encouragement. A person just naturally cannot help extending a man-to-man relation to all of those with whom he comes in contact. This, it seems to the speaker, has been the outstanding characteristic of every great leader in the world's history, of soldiers, rulers, teachers, preachers, and captains of finance and industry.

This does not imply that democracy and kindliness and modesty mean weakness and indecision, or that they are a substitute for ability, or for hard

work; but it does mean rather definitely that unless scientific skill, executive authority, and breadth of perspective are permeated with, controlled and directed by, love of one's fellows, they are "as sounding brass and a tinkling cymbal".

In speaking of love of one's fellows this should not be misunderstood. Mushy sentimentality, slackness, the avoidance of unpleasantness, are not evidences of kindliness, but rather of the most utter selfishness. No man who has made broad and intimate contacts with his fellows, whose eyes are open to their good qualities and to their defects, believes for one moment that they want, or will respond to, things that are not their due; rather, that justice curbing sympathy is what most men spontaneously respond to in leadership.

Above all else, the executive who knows men as individuals will demonstrate his fitness for humane leadership through faith in them. He will know that his associates are loyal to him in the exact measure that he is loyal to them; that his customers are fair in their relations to him proportionately as he is straight in his conduct to them. He will not assume himself to be possessed of virtues all his own but rather, realizing them as a common possession, will proceed on the thesis that most men respond in much the same spirit in which they are met.

And so, making his analyses from fact without bias, following principles of conduct rather than particular expediencies, having a proper perspective of his work in its relation to a humanity which he loves and trusts, and assured of the support of his co-workers, the executive (potential or actual) will have courage in simple, fundamental, straightforward decisions, recognizing that no one expects perfection of him and that he can afford to make an occasional mistake, granted only a straightforward willingness to recognize and to correct.

Now does not all this come fairly close to painting a picture of the essentials of a man as well as of an engineer? If so, the essential qualification of an engineer for any kind of a job, executive or otherwise, is that he be a man and work at it.

IRRIGATION DEVELOPMENT THROUGH IRRIGATION DISTRICTS

Discussion*

BY E. COURTLAND EATON AND FRANK ADAMS, MEMBERS, AM. SOC. C. E.†

E. COURTLAND EATON‡ AND FRANK ADAMS,§ MEMBERS, AM. SOC. C. E. (by letter).‖—The discussions by Messrs. Lyman,¶ Lippincott,** and Stout†† are much appreciated by the writers. These discussions have helped to illuminate a presentation that was necessarily much compressed. No divergent views have been advanced, but emphasis has been added to some of the characteristics of irrigation district development suggested or implied in the paper.

Mr. Lyman is correct in stating that there are disadvantages as well as advantages in the irrigation district. Nothing could be more true, and perhaps these disadvantages were not sufficiently stressed. The fact remains, however, that for the large majority of cases, the irrigation district in some form is the best type of irrigation organization now in use, a fact on which all, as far as the writers are aware, agree. In many situations it has fully and satisfactorily met the need and accomplished a high type of agricultural development. This can be said of all the fifteen Western States having irrigation district laws. In all these States, also, and probably within the last five years, some districts have been organized and some have issued bonds and built works in the absence of full justification—economic as well as physical. In between these two extremes most of the States having these laws can show some districts that are progressing only fairly well, either because of unfavorable physical conditions—scant water supply, mediocre engineering, and questionable soil—lack of settlers, or general agricultural depression. The gratifying thing, however, is that, given a situation that physically and economically justifies an irrigation enterprise, irrigation district laws and operations under them have so far advanced as to leave no question as to the adequacy of the irrigation district as the agency through which the development can successfully be carried forward.

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* Discussion on the paper by E. Courtland Eaton and Frank Adams, Members, Am. Soc. C. E., continued from October, 1926, *Proceedings*.

† Authors' closure.

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‖ Received by the Secretary, October 27, 1926.

¶ *Proceedings*, Am. Soc. C. E., April, 1926, Papers and Discussions, p. 800.

** *Loc. cit.*, p. 802.

†† *Loc. cit.*, October, 1926, Papers and Discussions, p. 1697.

One important feature of irrigation district development on which the discussion has not touched is the extent to which State authority should govern and carry moral responsibility in the organization and financing of irrigation districts and in carrying out their construction. Originally State authority was not involved. Districts were organized as men saw fit and most of them went merrily to their destruction without let or hindrance on the part of any agency of the public. This costly experience fully justified the amendments to the district laws which gave the States (usually through the State engineers) the right of at least partial veto on the organization of irrigation districts. Experience under this procedure has given still further justification and no one questions the practice. The next important forward step, which brought the State into the examination of construction plans and into the certification of irrigation district bonds for certain purposes, was also one that was warranted by the conditions that had developed out of costly experience and this, too, is fully accepted as sound procedure.

The specific questions that might profitably be raised about this whole matter of State authority in irrigation district development are:

(1) Conceding an adequate irrigation water supply, satisfactory soil and growing conditions, and a feasible plan of works, should the State withhold approval of organization or of bonding and construction programs if, in the opinion of the State official (or State officials) charged with responsibility for action, there is doubt as to the economic success of the project at the time proposed?

(2) Should the action of the State bond certification commissions with reference to irrigation district bond issues, when favorable, be of such nature as not virtually to constitute State "approval", since every "approved" project is not certain to succeed and since in the minds of most investors the "approval" now given carries at least moral State responsibility?

(3) Should the State, through appropriate administrative officials, hold a helpful and friendly attitude toward irrigation districts or proposed irrigation districts, particularly during the planning and construction period of organized districts, in order to insure a higher efficiency in the district management and to give it the benefit of the larger experience of the State?

(4) Should the State undertake to supervise the accounting of irrigation districts, both during and after construction, in the first instance to insure sound use of funds supplied by the investing public, and in the second instance to accomplish the highest efficiency in the management of these enterprises, which are really agencies of the State?

Since these questions were not raised in the paper or in the subsequent discussion, the writers will not argue them in their closing discussion. They feel free to state, however, that they would answer at least the last three of these questions in the affirmative.

In conclusion: The writers do not wish to advance the thought that the irrigation district as described in their paper is the final step in irrigation organization, although it is a permanently established form of organization for situations similar to the many in which it is successfully and satis-

factorily operating. Several other types of districts were referred to in the paper, the most important of which are the so-called "super" water-storage districts already formed and the conservation district under formation in San Joaquin Valley, California. These have been developed out of the necessity of building expensive storage works that embrace from two or three to fifteen lesser units that already have partial irrigation supplies, and each is organized around entire or practically independent stream systems.

There are some still larger problems of irrigation organization ahead, problems involving re-distribution of water from areas of excess to areas of deficiency and these, in turn, will demand even more comprehensive forms of organization and procedure. Whatever turn these take, however, they are not likely to depart very far from the principles of co-operation and community ownership and management exemplified in the irrigation district of to-day, except as wider State authority is found necessary to accomplish more economic utilization of still larger units of water and land, and as some financial participation by the State is required to bring the financial burden involved in these larger movements within a figure that the farmer on the land can pay.

CORROSION OF CONCRETE

Discussion*

By Messrs. E. E. R. Tratman and John R. Baylis†

E. E. R. Tratman,‡ Assoc. M. Am. Soc. C. E. (by letter).§—An examination of several concrete structures made a few years ago for the purpose of ascertaining the appearance of exposed concrete surfaces, is called to mind by the extended discussions of this paper, and of other recent papers on various aspects of the concrete problem.

The structures in question included railway and street bridges, retaining walls, river-front or bulkhead walls, and buildings of different kinds. Most of these structures were only a few years old, but only a very small proportion of them could be classed as satisfactory in appearance, owing to the development of craze marks and streaks and patches of discoloration which marred the surface. At several bridges and walls, water had seeped through the joints and the concrete mass, in spite of water-proofing on the back, and thus had added to the unsightly appearance. The defects noted were those inherent to the concrete, exclusive of the effects of dirt and smoke which may accumulate on any kind of surface. They were independent also of structural cracks, such as seem to be almost inevitable in long retaining walls. The examination was not concerned with structural disintegration of the concrete, but in some cases there may be a close relation between surface defects and general disintegration or corrosion of the concrete.

Although concrete buildings are being erected in large numbers, few of them have exposed concrete surfaces, a veneer of brick, stone, and terracotta being applied as a rule for the purpose of giving a pleasing and permanent finish. Such exterior coverings are sometimes cleaned or renovated at intervals. In other cases concrete surfaces are relieved by inserts of colored tile or by grooves and recesses to break up flat surfaces. Treatment of this latter kind, however, is utilized for decorative effect rather than to prevent or remedy a poor appearance of the finished concrete. The comments on the surface treatment of concrete by Mr. Newhall|| are in line with suggestions offered after the examinations mentioned, to the effect that it might be advisable to study the practical and economical possibilities of giving to exposed concrete a surfacing that would produce an appear-

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* Discussion on the paper by John R. Baylis, Assoc. M. Am. Soc. C. E., continued from December, 1926, *Proceedings*.

† Author's closure.

‡ Associate Editor, *Engineering News-Record*, Wheaton, Ill.

§ Received by the Secretary, November 8, 1926.

|| *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1813.

ance as good and as durable as that of a veneering with other material. Such treatment might result in considerable saving as compared with the usual veneer covering, while it would also avoid the unsightly appearance of structures which do not warrant the expense of such veneer.

Surface grinding and grouting of fresh concrete has been used in a number of cases, but does not appear to be an entire assurance against crazing and discoloration. Painting of exterior surfaces has been used less extensively and with rather doubtful results.

The two questions or problems involved are: First, the possibility of developing a treatment or manipulation of concrete that will give a surface of permanently good appearance with reasonable economy; and, second, the possibility of further developing such treatment for the protection of concrete exposed to water and wave action. As far as appearance is concerned the following appropriate statement* is quoted: "There is an open field for developing concrete construction that can honestly show its own face without causing pain because of its unsightliness."

JOHN R. BAYLIS,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The writer is grateful for the very interesting discussions of the subject, and feels that much of value has been added to the paper. There are only a few points where additional comment seems desirable. In most instances where there is disagreement with the writer's conclusions, it is believed to be due more to a misunderstanding of the subject than to differences of opinion. When more details of the problem of concrete deterioration are published it is believed that the value of some of the facts discussed in the paper will be more readily appreciated.

On a subject of this nature it is expected that there will be some differences of opinion. Dr. Bogue§ submits no evidence in his criticism, and the writer does not agree with his views on the points discussed. A criticism of this kind brings to mind a passage from a recent editorial in a well-known technical journal on the art of writing:

"No one has any right to expect that the writer's thoughts should agree with the reader's; it should suffice that the writer has given us his own thoughts expressed so far as possible in his own order and his own words. Here and there should be found a few lines worth reading, an idea or two worth considering, even a phrase which is illuminating or apposite."

It is gratifying to know that a few contractors are producing good concrete. The writer, however, does not agree with Mr. Ahlers‖ that the bad qualities of concrete should not have received greater consideration in the paper than the good qualities. He knows that a great deal of good concrete is being manufactured, and that there is much in the engineering literature telling of its good qualities. It is also known that there is much concrete that has been manufactured with the greatest care, that is deteriorating in strength. It was the writer's intention to point out a few reasons

* *Engineering News-Record*, December 16, 1920.

† Physical Chemist, Bureau of Eng., City of Chicago, Chicago, Ill.

‡ Received by the Secretary, November 19, 1926.

§ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1270.

‖ *Loc. cit.*, p. 1268.

for this deterioration, and it was assumed that the good qualities of concrete had already been fairly well advertised. It is true that some of the evils to which reference was made can be avoided by the use of the water-cement ratio, and this is a step in the right direction, but it is not the solution of all deterioration problems.

Mr. Ahlers, as well as Mr. Williams,* has failed to grasp the purpose of the void test. It was intended to be used in interpreting what to expect of concrete after it has been mixed, or more especially after it has been placed in the forms. No idea of the quantity of cement used can be obtained from the test as outlined. If it is desirable to know this it may be determined by carrying the test a little further, providing silica sand is used for the fine aggregate. Knowing the calcium content of the cement used, any variation of this constituent in the mortar sieved from the coarse aggregate is readily detected. Such a test is not reliable unless all aggregate larger than the No. 8 sieve size is removed, also if any aggregate finer than this size contains calcium soluble in acid. The writer has followed such a procedure for examining concrete in which the quantity of cement used was in doubt and finds that it gives fairly accurate results.

In his density test Mr. Williams assumes the volume and specific gravity of all materials used, to be known. It was the writer's intention that a testing engineer should collect samples as the concrete is being poured into the forms, or, even better, after it has been placed. In instances where it is possible to do so, it might be best to collect samples by cutting out small pieces of the concrete after it has set a day or more. Mr. Williams' method includes all voids, whereas that proposed by the writer does not include voids of more than about $\frac{1}{100}$ in. in diameter. Voids $\frac{1}{8}$ in. in diameter, or even larger, caused by air bubbles, do not materially affect the power of the concrete to resist internal chemical changes, yet such large voids may materially affect the total percentage of voids. Heretofore test methods have tried to measure all voids, a procedure which is practically useless for estimating the power of the concrete to resist the weathering agents.

The proposed void test is not an absolute measure of the durability of the concrete, but it seems to give better indications than almost any other test. Because a test is not absolutely positive in its indications does not signify that it is of no value. The measurement of aggregate by loose volume is not accurate, but it is certainly better than no measurement. With the same materials used and the same condition of exposure, the void test gives a better indication of the durability of the concrete than may be predicted by any other test. The flow of water through a piece of concrete may be influenced by so many factors that it is not a good criterion of the power of the concrete to resist deterioration. The tendency may be in the same direction in most instances, but the passage of water through the concrete is affected quite considerably by the tendency to form channels. In other words, practically all the flow may be passing through a very small section of the concrete.

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1505.

Those who are still of the opinion that all troubles can be eliminated by adherence to a certain water-cement ratio, or any other test that is made in advance of placing the concrete in the forms, will do well to read a recent article by Greene.* Attention is called to the following:

"Assume a column or wall, large enough for a man to work in. Place concrete in this wall of a consistency to give 1-in. slump at the mixer or as sampled out of a buggy. Run 7 ft. in depth of the concrete in three hours with men spreading the concrete. At the end of the run there will be loose water on top of the concrete and the material tested in the hole may show a slump of 4 in. * * * It is the judgment of the writer that samples for testing should be taken only in the hole and from the wettest concrete in the hole, not from the mixer or from the buggy. * * * The concrete should be bought 'f.o.b. the hole', not at the mixer, the dumping hopper or in the buggy."

The proposed void test would be very helpful for such instances as that described. It will show quite readily that the top concrete which has the excess water is too porous for durable concrete under many exposed conditions. Any test that takes the sample in advance of the last disturbance of the concrete in the forms will not give conditions as they are. Some kind of a test must be adopted that may be applied to concrete after it has been placed in the forms, or even after it has set, if engineers ever expect to get one indicative of actual conditions.

The new sieve size, to which Mr. McMillan† takes exception, is a necessity for the proposed void test, and it will be more helpful than the ¼-in. size in proportioning aggregate. It will be impossible to make an intelligent interpretation of the void test unless the quantity of aggregate larger than about $\frac{1}{10}$ in. in diameter is shown. For a test to be applicable to fairly small samples of concrete the dividing line between fine and coarse aggregate must be set at a point where there is practically no possibility of the particles separating from the mortar. It is a fact that ¼-in. aggregate will separate in much of the concrete now being manufactured especially with a little disturbance, such as working the concrete in the forms. The separation may be slight and not enough to affect the strength of the concrete in many instances, but it spoils the void test.

No one should attempt the void test unless the No. 8 sieve is used. A few samples from concrete after it has been placed, or after it has set, will impress one with the value of using this size.

In discussing the void test, Mr. McMillan makes the statement that, "all that the test accomplishes, therefore is, in effect, a determination of the quantity of combined water". How any one can conceive of this being the case, the writer is unable to understand. After being dried at room temperature (60° to 70° Fahr.) concrete will show considerably more than 50% of its total water absorption, except possibly for very dense mortar. The writer feels sure no one believes that the hydrated cement compounds are

* "Some Lessons Learned in Building a Long Concrete Bridge," by John F. Greene, *Engineering News-Record*, Vol. 97, No. 20, November 11, 1926.

† *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1819.

dehydrated at this temperature. Perhaps it would be helpful if both Dr. Bogue and Mr. McMillan made a few void tests in order that they might better understand its limitations.

It was not intended in this paper to cover the entire field of concrete deterioration. Porosity was discussed more as a means of aiding chemical changes than as an attempt to give the changes that actually are taking place. One of the most serious problems in concrete deterioration was not even mentioned. It is hoped that additional research work will be made public in the near future, and that this will be of some aid in leading to a clearer understanding of the limitations of Portland cement concrete.

STRAIGHT LINE PLOTTING OF SKEW FREQUENCY DATA

Discussion*

BY MESSRS. W. A. SHEWHART, W. H. R. NIMMO, AND JOHN TUCKER, JR.

W. A. SHEWHART,† ESQ. (by letter).‡—Some very broad claims are made for the method of analyzing data proposed by Mr. Goodrich. Hence it may be of interest to consider some illustrations in which these claims do not appear to be justified and then to consider a few general criticisms of the method. Both the case where the number of observations is small and where it is large will be considered.

Table 19 gives the results of six drawings of samples of 10 made under known conditions. In this case the quantity, X, corresponds to run-off in Mr. Goodrich's paper. The endeavor will be to find the best straight line to represent the distributions from which the samples were drawn. Figs. 18 and 19 show the corresponding ogives§ plotted on the special co-ordinate paper suggested by Mr. Goodrich.

TABLE 19.—VALUES OF X.

Sample 1.	Sample 2.	Sample 3.	Sample 4.	Sample 5.	Sample 6.
15	18	13	42	22	22
26	28	17	43	31	29
33	35	27	43	33	33
42	42	34	44	34	36
45	48	35	44	41	37
55	48	35	44	44	41
57	49	39	44	45	43
61	53	42	45	48	47
62	58	57	54	50	47
67	70	62	57	57	55

Samples 1 and 2 appear to fall approximately on the same straight line which is almost parallel to that representing Sample 3. This would indicate similar types of parent distributions with the median value for Samples 1 and 2 greater than that for Sample 3. In a similar manner Samples 5 and 6 appear to have come from the same parent distribution or universe, whereas the data for Sample 4 lie on a curve which is concave upward, indicating that

* Discussion on the paper by R. D. Goodrich, M. Am. Soc. C. E., continued from November, 1926, *Proceedings*.

† Bell Telephone Laboratories, Inc., New York, N. Y.

‡ Received by the Secretary, October 25, 1926.

§ An ogive curve is an integral curve.

they must have come from a universe different from that from which Samples 5 and 6 were drawn.

Now, the truth is that Samples 1, 2, and 3 were drawn from the same rectangular distribution,

$$y_1 = 2 \text{ for } 5 \leq X \leq 75;\ y = 0 \text{ elsewhere}$$

and Samples 4, 5, and 6 were drawn from a normal universe,

$$y_2 = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(X-m)^2}{2\sigma^2}}$$

in which, $m = 40$ and $\sigma = 10$. Hence, neither of the parent distributions should plot as a straight line on the paper suggested by Mr. Goodrich. The heavy curves in Figs. 18 and 19 show the integral curves for the two parent distributions. Obviously, the proposed graphical method does not give very reliable information in this case.

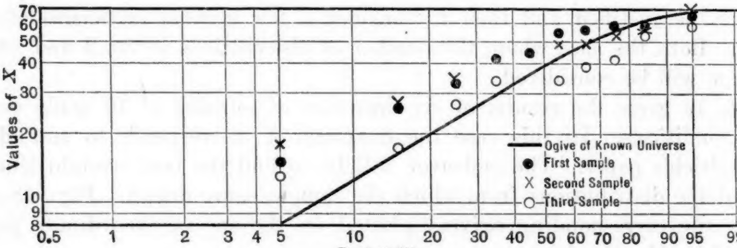

FIG. 18.—GRAPHICAL REPRESENTATION OF SAMPLES 1, 2, AND 3.

What will happen when the data are plotted on co-ordinate papers for which the observed curves should be straight lines? Of course, the ogives for the first three samples should plot as straight lines on ordinary co-ordinate paper, Fig. 20, and the other three should plot as straight lines on normal law probability paper, Fig. 21. Here again, however, we find what may be expected —samples of Size 10 are not sufficient to determine straight lines even when the right kind of co-ordinate paper is used.

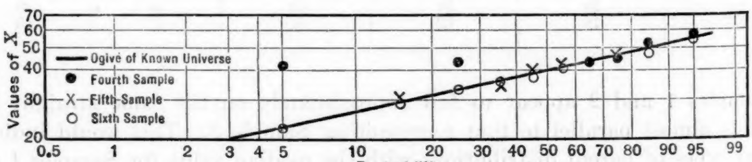

FIG. 19.—GRAPHICAL REPRESENTATION OF SAMPLES 4, 5, AND 6.

Similar instances of the failure of the method could be shown for samples as large as, or larger than, 50, hence it does not appear that the conclusion is justified, namely, that "the graphical analysis of 'integral skew frequency' or 'duration curves', as herein presented, is developed especially for the examination of records consisting of from 10 to 50 observations".

The author states that his method was developed to overcome some of the limitations of Pearson's method of fitting skew frequency distributions. For example, he makes specific mention of the fact that it is well recognized that Pearson's method breaks down for small numbers of observations, because the errors in the third and higher moments are so large. Now, the method under review is supposed to be especially adapted to the analysis of such short series of data, because a straight line can easily be found to represent the data. We already have seen certain cases where the method appears to break down. Furthermore, a little consideration shows that such a breakdown is to be expected because large errors of sampling exist in the estimates of the parameters required to fix the straight line on the special co-ordinate paper proposed by the author when n is small, just as large errors exist in the estimates of the parameters used to determine the shape of a frequency curve under similar conditions.

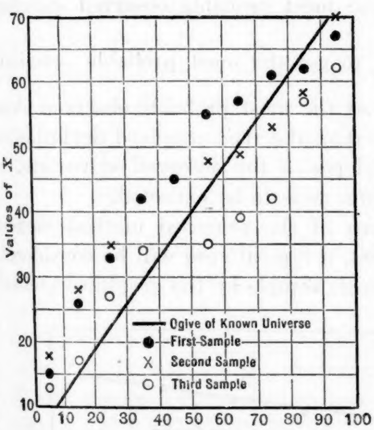

FIG. 20.—DATA OF FIG. 18 PLOTTED ON PAPER FOR WHICH THE OBSERVED POINTS SHOULD FALL ON THE HEAVY LINE.

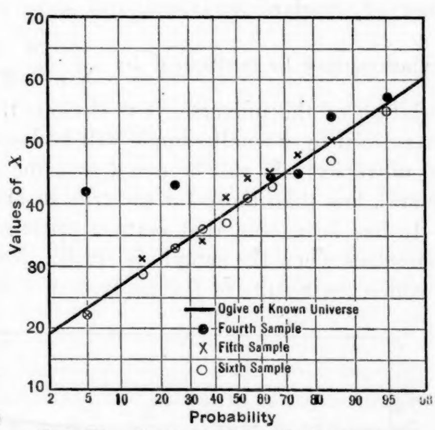

FIG. 21.—DATA OF FIG. 19 PLOTTED ON PAPER FOR WHICH THE OBSERVED POINTS SHOULD FALL ON THE HEAVY LINE.

One may go even further, however, than citing particular cases where the method proposed by Mr. Goodrich fails. Let us suppose that the type of the parent distribution is known and all that the graphical method is required to do is to give an estimate of the parameters. Let us find the best straight line representing a series of n observed data drawn from a normal universe,

$$y_3 = \frac{1}{\sigma' \sqrt{2\pi}} e^{-\frac{(X-m')^2}{2\sigma'^2}}$$

in which, m' and σ' are unknown. In order to determine the probability that a value of X will lie within the range, X to $X + dX$, estimates must be found of both the parameters, m' and σ'. Of course the median of the ogive is an estimate of m' and the semi-quartile range (used by Mr. Goodrich) divided by 2 (0.6745) is an estimate of σ'.*

* Of course the slope of the straight line drawn through the points representing the observed data is also an estimate of σ'

Now, it is well known* that the error of the median is 1.253 times as large as that of the arithmetic mean, $\sum_{i=1}^{n} \frac{X_1}{n}$, and the error of the previous estimate σ_1 of σ is 1.65 times as large as that of the customary estimate, $\sum_{i=1}^{n} \sqrt{\frac{(X_1 - X_{ar})^2}{n}}$, in which X_{ar} is the average value of X. Hence, the location of the center of the distribution (by the ogive method) in Equation (3)† by $1.57n$-observations is no better than that obtained by the arithmetic mean of n-observations or, in other words, the ogive method is only 63.7% efficient for the determination of the mean. In a similar manner the estimate of the standard deviation by the proposed method is only 36.8% efficient.‡

There is also another inherent error in the graphical method for the case in hand. Since the slope of the line on normal probability paper is given by the observed standard deviation, and since the most probable observed standard deviation must be multiplied by $\sqrt{\frac{n}{n-2}}$ to get the most probable standard deviation of the universe, it is obvious that the most probable observed slope determined by a small sample will be less than the true standard deviation of the universe.§ It will be noted that the slopes of the observed ogives are, in general, less than those for the true inverses, as is to be expected.

It has been seen that certain criticisms of the proposed method suggest themselves when the sample is small. Next, a typical case will be considered, in which the results of the analysis of a large sample by the graphical method

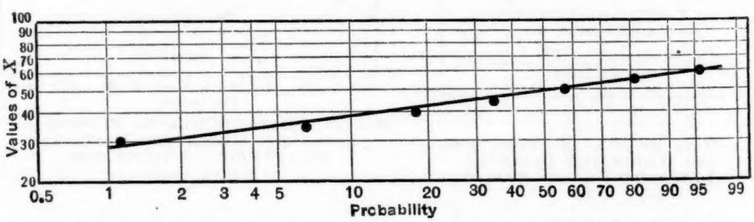

FIG. 22.—OBSERVED DISTRIBUTION OF A SAMPLE OF 15 050 OBSERVATIONS.

fails to give the desired information whereas standard methods of analysis prove satisfactory. The distribution of a sample of 15 050 observations is plotted in Fig. 22. The straight line appears to fit the data quite as well as some of the illustrations given by Mr. Goodrich and yet customary analytical methods show almost beyond a doubt that the observed data do not constitute a random sample The analysis of these data by customary methods has been published.‖

* "On the Probable Errors of Frequency Constants," by Karl Pearson, Biometrika, Vol. XIII, Pt. 1, pp. 113–132.

† *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1069.

‡ The efficiency of 64.9% could be obtained by estimating σ' from the ogive range of $\frac{100}{14}$ to $\frac{1\,300}{14}$ per cent.

§ "Correction of Data for Errors of Averages Obtained from Small Samples," by W. A. Shewhart, *Bell Technical Journal*, April, 1926.

‖ "Quality Control Charts," by W. A. Shewhart, *Bell Technical Journal*, October, 1926.

It is hoped that this discussion will help to point out some of the pitfalls in attempting to draw conclusions from small samples. In the light of this discussion it also seems that several of the broad claims for the proposed graphical method should be somewhat modified. It also appears that much more work must be done before the new method can be expected to replace the more reliable standard analytical methods either for small or large samples.

W. H. R. Nimmo,* Assoc. M. Am. Soc. C. E. (by letter).†—This paper is a valuable addition to the previous papers on the same subject by other members of the Society. The comparatively simple method of treating frequency data developed by the author should prove of great value to hydraulic engineers in the solution of their problems. The writer has been able to make immediate practical application of the author's methods to his own work.

Early in 1918 the Government of Tasmania decided to develop the full capacity of the Great Lake for the generation of power. This lake has a catchment area of 153 sq. miles, including the water surface of 42 sq. miles at the natural winter level. By constructing a dam about 1 200 ft. long at the outlet at the south end of the lake, it was possible to obtain ample storage capacity to completely regulate the run-off from the lake and also from streams which could be diverted into it. The lake is at an elevation of 3 300 ft., and parts of the catchment basin rise to more than 4 000 ft. above sea level.

A more or less complete record of rainfall had been kept for twenty-eight years at the dam site. Owing to frequent snowfalls the country is not continuously inhabited and the rainfall observations at other places in the drainage area were limited to partly complete records of less than two years' duration. It was known, however, that the precipitation at the north end of the area was from two to three times that at the dam site. Records of evaporation from a 3-ft. land pan, situated near sea level at Hobart, distant 80 miles from the lake, were available for eight years. Daily records of the level of the lake and the discharge therefrom were available for one complete year only, and no continuous gaugings had been made of any other stream in the State.

It fell to the lot of the writer to determine, from this meager data supplemented by gaugings of rivers on the mainland of Australia, the probable total mean annual run-off from the lake and the storage capacity required to regulate it completely. The investigation resulted in the development of a formula for the annual run-off indicating that the maximum continuous yield would be 235 000 acre-ft. per year, equivalent to 324 sec-ft. To provide for the storage of water diverted from adjacent areas, a dam has been constructed to impound 35 ft. of water, creating a storage of 1 150 000 acre-ft. having a water surface of 60 sq. miles at full supply level.

As the records of run-off for each year since 1918 have become available, it has been a source of much interest to the writer to compare his estimated

* Civ. Engr., Metropolitan Water Supply and Sewerage Board, Brisbane, Queensland, Australia.

† Received by the Secretary, November 15, 1926.

discharges with the actual outflow from the lake. Records for nine years are now available, including 1925 which was a year of very low run-off, and for this period the estimated yield exceeds the measured run-off by 16 per cent. The data for applying the author's method to the 9-year record of run-off are given in Table 20 and the corresponding curves are

TABLE 20.—RUN-OFF OF THE GREAT LAKE, TASMANIA.

Year. (1)	Run-off, in thousands of acre-feet per year, R. (2)	Percentage of time, t. (3)	$R - 120$. (4)
1925	134	5.5	14
1918	153	16.7	33
1919	156	27.8	36
1920	166	38.9	46
1922	205	50.0	85
1921	218	61.2	98
1924	308	72.2	188
1923	348	83.3	228
1917	356	94.5	236

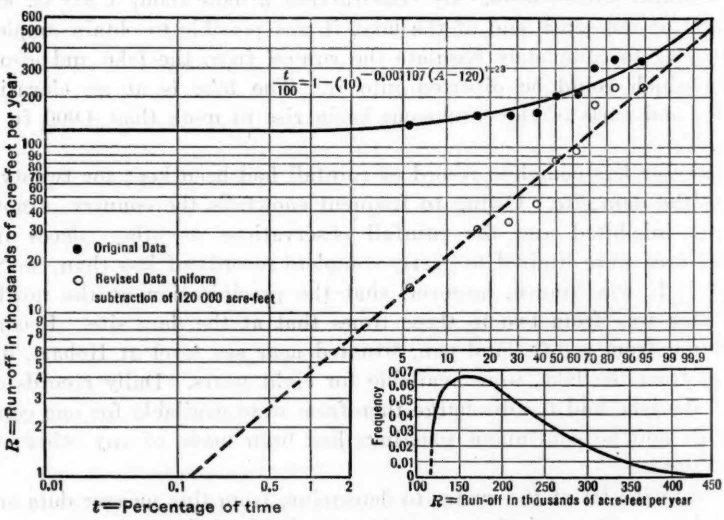

FIG. 23.—CURVES FOR GREAT LAKE, TASMANIA.

shown on Fig. 23. From the curves the following values are obtained, namely, $a = 120$, $c = 1.23$, and $h = 0.001107$. Substituting these values in the author's Equation (5),* the probable mean annual run-off is found to be 238 000 acre-ft. per year, equivalent to 328 sec-ft., which agrees closely with the writer's estimate of 324 sec-ft.

JOHN TUCKER, JR.,† ESQ. (by letter).‡—The general subject of the study of statistical frequency data is an interesting one and apparently is becoming of more and more use to the engineer. Any new and original methods

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1070.
† Washington, D. C.
‡ Received by the Secretary, December 2, 1926.

of the analysis of the data as those proposed by the author are of interest and may prove of considerable value. There are, however, some criticisms to be made of the paper.

The coefficient of variation is defined as $A \sqrt{\dfrac{\Sigma v^2}{n - 1}}$ and must always be of one sign, chosen positive for convenience. A negative value is, therefore, as meaningless as a negative radius of gyration. The negative values either indicate a defective method of determination or that the author's coefficient of variation is a new parameter which differs from the accepted quantity. The coefficient of variation is the parameter most readily and reliably determined, and hence the negative values as given by the author* are not within the range of possible error by reason of the precision of their determination. They indicate to the writer a lack of reliance to be placed in this method of determination.

The author objects to the use of the Pearson frequency curves because of the uncertainty of the accuracy of the constants when based on coefficients computed from a series of not more than twenty or thirty terms. It should be carefully noted that the curves obtained by the author will not approximate the curve attained from an infinite universe (that is, a very large amount of data) any closer than the Pearson curves obtained from the same data. The precision of the Pearson parameters, β_1 and β_2, may be expressed by their standard deviation.† The probability of the data indicating any of the Pearson types of curves is, therefore, definitely determined. Mr. Goodrich does not express the precision of any of the results obtained by his methods.

The author calls for the assumption of an upper or lower limit to the curve.‡ This method, of course, invalidates the usefulness of the analysis, and calls for an extrapolation process. It is almost unnecesary to state that this is the most undependable and dangerous process to apply in mathematics.

Referring to the numbered "Conclusions"‡ given by the author:

1.—It is not necessary to limit the number of observations in graphical plotting. As stated by the author, although the individual points will not appear distinctly, the curve is indicated with greater accuracy, the greater the number of points.

3.—The use of the graphical methods developed by the author are of very small value in statistical studies or in any application where extrapolation is applied. Extrapolation is always dangerous; in the present application the danger is removed by assuming a maximum limit, but the value and force of the method are thereby destroyed. Examination of several of the diagrams will show the indefiniteness of the curve form. In Fig. 8§ neither branch is definitely indicated, the left branch being more definite than the right. In Fig. 9‖ the left branch should be a constant frequency,

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1073.
† "Statistical Methods for Research Workers" (Biological Monograph), by R. A. Fisher.
‡ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1104.
§ *Loc. cit.*, p. 1085.
‖ *Loc. cit.*, p. 1087.

not the continuation of the given straight line. The right-hand branch of Fig. 17* is another forceful illustration.

4.—The reason for assuming that extrapolation by means of the author's graphical straight-line methods is more reliable than extrapolation of a curve of double curvature as implied by him, is not very apparent, especially since the plotted points at the ends of several of the curves indicate a trend away from the plotted straight line.

5.—Frequency curve paper may be used with equal effect and with equally satisfactory results as the logarithmic or skew frequency paper, in the writer's opinion.

6.—It is true that any engineer may follow and apply the methods of the paper, but it is absurd to claim that he can make scientific investigation of frequency data by any method unless he has had a rather rigorous training in statistical mathematics. In no other branch of elementary mathematics is error in methods and in application so imminent.

7.—Statistical methods are developed for the best possible determination of unknowns. If the results depend on the skill and judgment of the engineer then the method loses its force and value; thus skill and judgment become the sole necessity, and the statistical methods should be discarded.

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* *Proceedings*, Am. Soc. C. E., Papers and Discussions, p. 1100.

DISTRIBUTION OF REINFORCING STEEL IN CONCRETE BEAMS AND SLABS

Discussion*

BY MESSRS. CHARLES S. WHITNEY AND DAVID A. MOLITOR

CHARLES S. WHITNEY,† M. AM. SOC. C. E. (by letter).‡—The principles given in this paper appear to be based on erroneous premises. They are contrary to generally accepted practice and the burden of the proof, therefore, should be on the author. Both experimental and theoretical investigations have established that the theory of elastic frames is applicable to reinforced concrete construction, and that it is the most practical method of design. For the results of the application of this theory the author would substitute arbitrary rules which give very different results.

In developing his rules, Mr. Myers has made two fundamental errors: First, the variation of the moments of inertia in T-beams does not prevent the satisfactory application of the theory of elasticity; and, second, in establishing bending moment coefficients, he has considered only the case of full load on all spans. With partial loading, the sum of the maximum moments at the center and supports may exceed the simple span moment.

Ten or fifteen years ago some designers were using such rules as those suggested by the author with apparent success; but that does not indicate that structures so designed would prove as efficient under full load as those designed in accordance with the latest rulings of the Joint Committee on Specifications for Concrete and Reinforced Concrete.

DAVID A. MOLITOR,§ M. AM. SOC. C. E. (by letter).‖—The author has presented a subject that deserves careful consideration because it deals with several of the mooted questions of designing reinforced concrete beams with restrained ends as these questions arise in the realm of building construction.

For concrete beams of rectangular cross-section, simply supported, but continuous over their supports, and subject to variable moving loads, the Theorem of Three Moments, which is based on the Theory of Elasticity, will undoubtedly lead to the most economic designs. Such beams may be used in smaller bridges, but rarely find application in monolithic building construction where most beams have a T-shape.

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* Discussion of the paper by Boyd S. Myers, M. Am. Soc. C. E., continued from November, 1926, *Proceedings*.

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‡ Received by the Secretary, October 15, 1926.

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‖ Received by the Secretary, October 25, 1926.

The T-shaped beam in buildings is strictly monolithic with columns and floor-slabs, and reinforcing steel so interlocked as to prevent unsightly cracks. The superimposed loads are of extremely variable character, uniformly distributed dead and live loads, and various concentrated loads, all of which may attain the maximum values assumed in the design, or they may be greatly reduced when there is no live load. It would be futile to attempt a rigid analysis of such a beam, or system of beams, in accordance with the theory of elasticity, and hence the problem is usually solved by some approximate method.

The earlier attempts in this direction were naturally patterned after the idea of taking account of restraint, first, of the beams themselves where they are continuous over supports, and, second, by fixity at column connections. This was prompted by the appearance of cracks whenever inadequate steel reinforcement was provided at points of restraint.

Since the live load may be applied to any spans, or portions of spans, of a continuous system of beams for the purpose of design, the following approximate moments were used, $\frac{w\,l^2}{10}$ for the positive center and negative end moments of an end span (restrained at one end only), and $\frac{w\,l^2}{12}$ for the positive center and negative end moments of all interior spans (restrained at both ends). These results follow from an application of the Theorem of Three Moments for maximum load effects, assuming a uniform moment of inertia throughout. The maximum sum of the end and center moments of an interior span is thus $\frac{w\,l^2}{6}$, while the theoretical minimum for this sum is only $\frac{w\,l^2}{8}$ for a case of simultaneous loading over all spans. For end spans this sum is $\frac{w\,l^2}{5}$, while in reality it is not a constant, but must vary depending on the point of contra-flexure. According to this method of design the end moments at a column, between an end span and an adjacent interior span, are usually unequal, so that such a column itself would be subjected to bending.

While this rule is simple when applied to rectangular beams, for which it was actually intended, difficulties are encountered in its application to sections of T-form. The T-beam is uneconomical when designed for equal end and center moments because, for a correctly designed section at the center, where the flange (slab) is adequate for the compression area and the stem and steel are just sufficient for the tensile stress, the end section of the beam will be incapable of taking the compression on the lower side which now falls to the stem area, and compressive steel must be provided or the stem must be haunched. This difficulty can be avoided by reducing the end moments and increasing the center moments. Thus, the portions subjected to positive moment will be lengthened to develop the available concrete on top, and the portions subjected to negative moment will be shortened as limited by the available concrete in the stem only.

Now, if these moments are combined in some arbitrary manner, at the same time retaining a sum moment of $\frac{w\,l^2}{6}$, for an interior span, the laws of continuity as established by the elastic properties of the materials will be violated unless due account is taken of this violation in the design of the beam. This may best be accomplished by ignoring the theoretical moment distribution indicated for continuity and treating the chain of spans as a series of cantilever systems in which the several points of contra-flexure may be regarded as arbitrarily assigned hinged points; but unless hinges are actually provided at these points of contra-flexure, such cantilever systems may not act in accordance with the assumptions, and the position of each hinge may change to accommodate itself to the various positions of the live load on the several spans.

Take, for example, any interior span having a certain definite loading and restrained by known end moments from adjacent spans. Then, the points of contra-flexure are at once fixed and the corresponding center moment is determined, making the sum of this center moment and the mean of the two end moments (if unequal), equal to the theoretical minimum of $\frac{w\,l^2}{8}$. Now, reduce the end moments by unloading the side spans, and the points of contra-flexure will shift accordingly to new positions, resulting in a larger center moment but again making the sum of the center moment and the mean of the two new end moments equal to $\frac{w\,l^2}{8}$. Thus, within limits of possibility in shifting live loads, the center moment will attain a maximum value when the corresponding end moments are minima; and, conversely, the center moment will become minimum when the two end moments are maxima. Now the sum of the maximum values of the end and center moments is necessarily greater than $\frac{w\,l^2}{8}$.

Since a beam must be capable of sustaining the maximum load effects at all points with equal safety, therefore, the author is seriously in error when he bases his design on the minimum sum of $\frac{w\,l^2}{8}$, which could be correct only for a single simultaneous condition of uniform loading over all spans.

The question then resolves itself into assigning a reasonable, but safe, value to the sum of center and mean end moments of an interior span, such that the shift in points of contra-flexure may not exceed permissible limits. Obviously the value, $\frac{w\,l^2}{8} = 0.125\,w\,l^2$, is too small and the value, $\frac{w\,l^2}{6} = 0.167\,w\,l^2$, is rather large because it is based on a ratio of live load to dead load that is quite out of proportion for buildings and was originally chosen to fit bridge loadings with relatively high live loads.

The 1919 Building Code of the City of Detroit, Mich., specifies that, for beams supported partly or entirely by cantilever action, the sum of the

negative and positive bending moments, taken together, shall not be less than $\frac{6}{5} = 1.2$ of the bending moment for non-continuous members, and the assumed points of inflection of such special cantilever construction shall be clearly marked. This practice has been followed by progressive engineers in Detroit during the past ten years.

Since the code does not distinguish between end and interior spans, it must be presumed that the 20% increase applies to both. Also, since this increase is applied to the sum of the end negative and center positive moments, it must apply, therefore, to each of the two moments separately. Accordingly, for interior spans, the sum of the negative and positive moments becomes $1.2 \times 0.125\ w\ l^2 = 0.15\ w\ l^2$. For end spans this sum is not a constant but increases as the point of contra-flexure is placed farther out from the restrained end. The 20% increase for end spans is, therefore, even more necessary than for interior spans.

The subject is here presented in simple form for practical use according to the writer's views, giving formulas (see Fig. 17 and Table 1) and a table of moments (Table 2). The formulas on the left-hand side of Table 1 were derived for cantilever spans having all spans simultaneously loaded with a uniform load, w, per foot of span. The 20% increase was then applied (right-hand side of Table 1) to allow for variations in the points of contra-flexure as affected by different cases of loading, and the formulas thus obtained were used to compute Table 2 giving end and center moments.

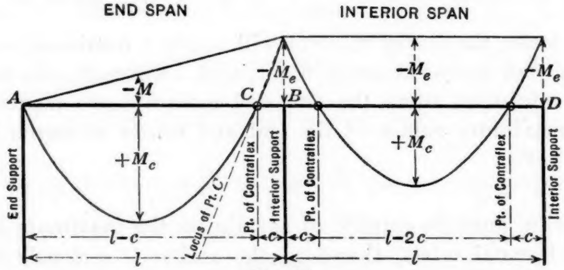

FIG. 17.

It should be noted that the locus of the point of contra-flexure for an interior span is the parabola representing the constant sum moment, $0.15\ w\ l^2$, while for an end span the locus of the point, C, is a straight line (Fig. 17) such that for every point, c, there will be a separate parabola passing through it to represent the end and center moments.

Fig. 17 is drawn for one particular set of moments, namely, $M_c = 0.08\ w\ l^2$ and $M_e = 0.07\ w\ l^2$, with $c = 0.135\ l$ for the interior span; and $M_c = 0.117\ w\ l^2$ and $M_e = 0.07\ w\ l^2$, with $c = 0.117\ l$ for the end span. This makes M_e equal for both spans, and no moment is transmitted to the column between the two spans. The same feature may be preserved even when the spans are of unequal lengths or when the loads are widely different on the two spans, as will be illustrated by a problem.

Table 2 is self-explanatory and will suffice for all spans of variable lengths and all cases of loading. The numerical moment coefficients, $\frac{1}{m}$, are to be multiplied by $w\,l^2$ to give actual moments, and the values of $\frac{c}{l}$ for points of contra-flexure are to be multiplied by l to give the actual distances, c.

TABLE 1.—FORMULAS FOR CANTILEVER BEAMS, UNIFORMLY LOADED (SEE FIG. 17).

MOMENTS FOR ALL SPANS LOADED. INTERIOR SPANS.	MOMENTS INCREASED 20%. INTERIOR SPANS.
$M_c = \frac{w}{8}\left(l - 2c\right)^2$	$M_c = 0.15\,w\,(l - 2c)^2$
$M_e = \frac{w\,c}{2}\left(l - 2c\right) + \frac{w\,c^2}{2} = \frac{w\,c}{2}\left(l - c\right)$	$M_e = 0.6\,w\,c\,(l - c)$
$M_c + M_e = \frac{w\,l^2}{8} =$ constant	$M_c + M_e = 0.15\,w\,l^2 =$ constant
$c = \frac{l}{2} - \sqrt{\frac{l^2}{4} - \frac{2\,M_e}{w}} = \frac{l}{2} - \frac{1}{2}\sqrt{\frac{8\,M_c}{w}}$	$c = \frac{l}{2} - \sqrt{\frac{l^2}{4} - \frac{M_e}{0.6\,w}} = \frac{l}{2} - \frac{1}{2}\sqrt{\frac{M_c}{0.15\,w}}$
END SPANS.	END SPANS.
$M_c = \frac{w}{8}\left(l - c\right)^2 = \left(1 - \frac{c}{l}\right)^2 \frac{w\,l^2}{8}$	$M_c = 0.15\left(1 - \frac{c}{l}\right)^2 w\,l^2$
$M_e = \frac{w\,c}{2}\left(l - c\right) + \frac{w\,c^2}{2} = \frac{w\,c\,l}{2}$	$M_e = 0.6\,w\,c\,l$
$M = \frac{M_e}{2\,l}\left(l - c\right) = \frac{w\,c}{4}\left(l - c\right)$	$M = \frac{0.6\,M_e}{l}\,(l - c) = 0.3\,w\,c\,(l - c)$
$M + M_c = \frac{w}{8}\left(l^2 - c^2\right)$	$M + M_c = 0.15\,w\,(l^2 - c^2)$
$M_c + M_e = \frac{w}{8}\left(l + c\right)^2 =$ variable	$M_c + M_e = 0.15\,w\,(l + c)^2 =$ variable
$c = \frac{M_e}{0.5\,w\,l}$	$c = \frac{M_e}{0.6\,w\,l}$

Now it may be contended that, because the points of contra-flexure are treated as imaginary hinges of a cantilever system, which hinges have not actually been provided in the construction, it is not certain that the structure will behave in accordance with the assumptions. This point can be answered as follows: In the first place, any of the various assumptions that may be made as to moment distribution in this analysis represents possible conditions for some cantilever system, and if the material (both steel and concrete) at any section of a beam is ample to care for the moment assigned to it, each such section must at least do its share of the total work to be performed. Furthermore, the possible shift of a point of contra-flexure can be reduced to a minimum limit by bending up part of the positive steel so as to have the first bent bar intersect the beam axis in the point of contra-flexure. Some negative steel must also extend out from the support to the adjacent points of contra-flexure and for a distance of at least $\frac{d}{2}$ beyond these points.

TABLE 2.—End and Center Moments for Continuous Beams.

The center moment need never exceed $\frac{w\,l^2}{8}$ for any span.

For interior spans the center moment shall be not less than $\frac{w\,l^2}{16}$, and the end moment shall be not less than $\frac{w\,l^2}{40}$.

For end spans the end moment shall be not less than $\frac{w\,l^2}{20}$, and may be anything, depending on adjacent spans.

The quantity, m, is the coefficient in the moment equation, such that $M = \frac{1}{m}\,w\,l^2$.

End Spans.					Interior Spans.				
End Moment.		Center Moment.		Values of $\frac{c}{l}$.	End Moment.		Center Moment.		Values of $\frac{c}{l}$.
Values of m.	Values of $\frac{1}{m}$.	Values of m.	Values of $\frac{1}{m}$.		Values of m.	Values of $\frac{1}{m}$.	Values of m.	Values of $\frac{1}{m}$.	
19.2	0.0522	8	0.1250	0.087	40	0.0250	8	0.1250	0.045
18.5	0.0540	8.05	0.1242	0.090	35.1	0.0285	8.25	0.1215	0.050
17.5	0.0570	8.15	0.1229	0.095	32.0	0.0312	8.40	0.1188	0.055
16.7	0.0600	8.25	0.1215	0.100	29.4	0.0340	8.63	0.1160	0.060
15.9	0.0630	8.33	0.1202	0.105	27.4	0.0365	8.82	0.1135	0.065
15.1	0.0660	8.43	0.1188	0.110	25.6	0.0391	9.03	0.1109	0.070
14.5	0.0690	8.52	0.1175	0.115	24.1	0.0416	9.23	0.1084	0.075
13.9	0.0720	8.62	0.1162	0.120	22.7	0.0440	9.45	0.1060	0.080
13.3	0.0750	8.73	0.1148	0.125	21.5	0.0467	9.68	0.1033	0.085
12.8	0.0780	8.82	0.1135	0.130	20.3	0.0492	9.93	0.1008	0.090
11.9	0.0840	9.04	0.1109	0.140	19.4	0.0516	10.2	0.0984	0.095
11.1	0.0900	9.24	0.1084	0.150	18.5	0.0540	10.4	0.0960	0.100
10.4	0.0960	9.38	0.1058	0.160	17.7	0.0564	10.7	0.0936	0.105
9.80	0.1020	9.69	0.1033	0.170	17.0	0.0587	10.9	0.0913	0.110
9.25	0.1080	9.92	0.1009	0.180	16.4	0.0610	11.2	0.0890	0.115
8.78	0.1140	10.15	0.0985	0.190	15.7	0.0634	11.5	0.0866	0.120
8.33	0.1200	10.4	0.0960	0.200	15.3	0.0655	11.8	0.0845	0.125
6.67	0.1500	11.8	0.0845	0.25	14.7	0.0678	12.1	0.0822	0.130
5.55	0.180	13.6	0.0735	0.30	14.3	0.0700	12.5	0.0800	0.135
4.76	0.210	15.7	0.0635	0.35	13.8	0.0723	12.9	0.0777	0.140
4.16	0.240	18.5	0.0540	0.40	13.4	0.0744	13.2	0.0756	0.145
3.70	0.270	22.0	0.0454	0.45	13.1	0.0765	13.6	0.0735	0.150
3.33	0.300	25.7	0.0375	0.50	12.7	0.0786	14.0	0.0714	0.155
2.78	0.360	41.6	0.0240	0.60	12.4	0.0805	14.4	0.0695	0.160
2.38	0.420	74	0.0135	0.70	12.1	0.0828	14.9	0.0672	0.165
2.08	0.480	167	0.0060	0.80	11.8	0.0846	15.3	0.0654	0.170
1.85	0.540	666	0.0015	0.90	11.6	0.0866	15.8	0.0634	0.175
1.67	0.600	∞	0.0000	1.00	11.4	0.0875	16.0	0.0625	0.177

Also, since the loads applied to a beam will first stress the central portion containing the positive steel before the load effect can reach the supports, it follows that the positive steel must work to capacity, delivering to the cantilever ends only that portion of the load in excess of its capacity and for which the negative steel makes ample provision.

On the other hand, enough negative steel may be provided to carry the entire load on cantilever arms, thus entirely unloading the central portion where little or no positive steel would then be required. However, this alternative is not utilized except in special cases of short interior spans or short end spans when the end moments from the longer adjacent spans are proportionately large.

The following example is given to illustrate the method of designing cantilever beams in accordance with this outline. Three tables (not here reproduced) are useful in designing concrete beams. They are (1) table of moments of resistance ($M\,R$) of rectangular concrete beams for various widths and depths according to the formula, $M\,R = \frac{R_c\, b\, d^2}{12}$ ft-lb., in which, the width, b, and the depth, d, are in inches, and $R_c = \frac{1}{2}\, f_c\, k\, j$; (2) table of moment of resistance per 1 in. width of **T** for various effective depths of beam; and (3) moment of resistance in foot-pounds for 1 sq. in. of compression steel for various effective depths of a beam.

Example.—Three spans have uniform loads of 2 200 lb. per lin. ft., and concentrated loads as shown in Fig. 18. Assume that $f_c = 750$ lb. per sq. in.; $f_s = 20\,000$ lb. per sq. in. for longitudinal steel and 16 000 lb. per sq. in. for stirrups; $n = 15$; $R_c = \frac{1}{2}\, f_c\, k\, j = 118.8$; and $M\,R = \frac{118.8}{12}\, b\, d^2$ ft-lb. for concrete sections. The shear on the reinforced section may be as much as 120 lb. per sq. in. The beam has a 4-in. solid slab flush with the top and will act as a **T**-beam.

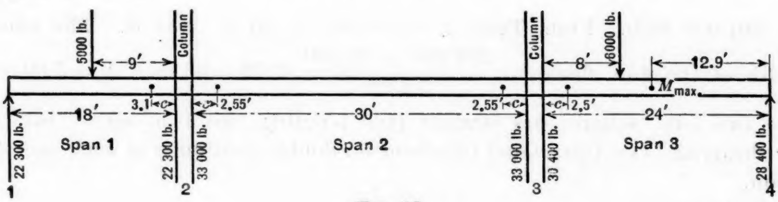

Fig. 18.

Maximum moment, Span 1:

$$2\,200 \times \frac{18^2}{8} + 5\,000 \times \frac{18}{4} = 111\,800 \text{ ft-lb.};\ w\,l^2 = 8 \times 111\,800 = 894\,000 \text{ ft-lb.}$$

Maximum moment, Span 3:

$$28\,400 \times 12.9 - 2\,200 \times \frac{12.9^2}{2} = 184\,000 \text{ ft-lb.};\ w\,l^2 = 8 \times 184\,000 = 1\,472\,000 \text{ ft-lb.}$$

For Span 2:

$$w\,l^2 = 2\,200 \times 30^2 = 1\,980\,000 \text{ ft-lb.}$$

First, choose a beam section adequate for the maximum end shear which occurs in the center span (Span 2). For a shear of 33 000 lb. make the beam 12 by 30 in., giving

$$v = \frac{33\,000}{0.88 \times 12 \times 28} = 112 \text{ lb. per sq. in.}\quad M\,R = \frac{118.8}{12}\, b\, d^2 = -93\,100 \text{ ft-lb.}$$

for the stem, without using any compression steel. This moment requires a tension steel area, $A_s = \frac{M \text{ ft-lb.}}{1\,460\, d} = \frac{93\,100}{1\,460 \times 28} = 2.27$ sq. in. Use three 1-in. round bars.

This section will be accepted for all spans, making all end moments equal to 93 000 ft-lb., and the corresponding center moment for each span will be found, using the moment coefficients given in Table 2.

End Span 1.—For the end moment,

$$M_e = \frac{w\,l^2}{m}, \text{ or } 93\,000 \text{ ft-lb.} = \frac{894\,000}{m}$$

From this, $m = 9.65$. For this value of m for M_e, the value of m for M_c will be 9.75. Hence, $M_c = \frac{894\,000}{9.75} = 92\,500$ ft-lb. Again using Table 2, $c = 0.172 \times 18 = 3.1$ ft. Since M_c is less than M_e, no **T** is required at the center. $A_s = \frac{92\,500}{1\,460 \times 28} = 2.26$ sq. in. Use three 1-in. round bars. Of these three 1-in. round bars, make one straight (full length); one bent for shear; and one bent for double continuity at the column end of the beam.

Intermediate Span 2.—

$$M_e = \frac{w\,l^2}{m} = \frac{1\,980\,000}{21.3} = 93\,000 \text{ ft-lb.}$$

Thus, for M_e, m is 21.3; for M_c, it will be 9.7. Hence, $M_c = \frac{1\,980\,000}{9.7} = 204\,000$ ft-lb. From Table 2, $c = 0.085 \times 30 = 2.55$ ft. The effective width of the 4-in. flange $= \frac{204\,000 - 93\,000}{5\,250^*} + 12 = 33$ in. $A_s = 5.00$ sq. in. Use two 1-in. square bars straight (full length); two 1-in. round bars bent for shear; and two 1-in. round bars bent for double continuity at each end of the beam.

End Span 3.—

$$M_e = \frac{w\,l^2}{m} = \frac{1\,472\,000}{15.85} = 93\,000 \text{ ft-lb.}$$

Thus, for M_e, m is 15.85; for M_c, it will be 8.33. Hence, $M_c = \frac{1\,472\,000}{8.33} = 177\,000$ ft-lb. From Table 2, $c = 0.105 \times 24 = 2.5$ ft. The effective width of the 4-in. flange $= \frac{177\,000 - 93\,000}{5\,250^*} + 12 = 28$ in. $A_s = 4.33$ sq. in. Use two 1-in. square bars straight (full length); two 1-in. round bars bent for shear; and one 1-in. round bar bent for double continuity at the column end of the beam.

The bars bent for double continuity are bent up at the point of contra-flexure at 45° and extend into the adjacent spans 1 ft. beyond the respective points of contra-flexure nearest the column support. It will be seen that the foregoing arrangement of bent bars furnishes three 1-in. round bars over the top of each support as required for the end moments of 93 000 ft-lb. The bars bent for shear are bent up at 45° next to the columns and at the outer ends. Likewise, they are hooked into the columns and at the outer ends.

* 5 250 is the *MR* for 1-in. width of a 4-in. flange on a beam of 28-in. effective depth, which may be found in standard tables.

No extra bars are thus required for the negative end moments as the necessary steel was obtained by proper bending and careful selection of bar sizes to make the steel areas correspond with the computed areas at all beam sections. Also, the end moments on each side of each column being equal, there will be no moment transmitted to the columns except a very nominal amount which may result from partial live loads on any of the spans.

In the foregoing example, say, 2.0 sq. in. of compression steel might have been utilized by extending the bottom straight bars into the columns and adjacent spans, making the negative end moments 129 000 ft-lb. instead of the 93 000 ft-lb. furnished by the concrete stem only. This would have resulted in somewhat smaller center moments and possibly a slight economy in steel.

If, for any reason, the end moments of an interior span are not made equal, a condition which is governed by the lengths and loadings of the adjacent spans, then the center moment is based on the mean value of the two unequal end moments. Similarly, if a certain end moment is assigned at Column 1 or Column 4, to be transmitted from the end spans to the end columns, then such end spans are designed for center moments based on the average end moments of these spans, the same as for interior spans with unequal end moments.

The shear reinforcement still to be provided in the form of stirrips is computed on the assumption that the concrete is permitted to take 40 lb. of shear per sq. in., and the bent shear bars are good for the added shear increment between the point of the first bend and the end of the span.

The gross end shear in the center span (Span 2) is 33 000 lb.; the stem area then takes 40 $bjd = 11\,760$ lb. and the bent bars will carry $3 \times 2\,200 = 6\,600$ lb., leaving a net shear, $V = 14\,640$ lb., to be taken by stirrups.

The minimum stirrup spacing is now found from the formula, $s = \dfrac{f_s\, A_s\, j\, d}{\text{net } V}$, in which, $f_s = 16\,000$ lb. per sq. in; $j = 0.88$; d = effective depth of beam; and A_s = area of one stirrup = 0.22 sq. in. for $\frac{3}{8}$-in. round, 0.39 sq. in for $\frac{1}{2}$-in. round, and 0.61 sq. in. for $\frac{5}{8}$-in. round V-stirrups.

Hence, for Span 2:

$$s = \frac{3\,080\, d}{V} \text{ for } \tfrac{3}{8}\text{-in. round } V\text{-stirrups} = 6 \text{ in.}$$

$$s = \frac{5\,500\, d}{V} \text{ for } \tfrac{1}{2}\text{-in. round } V\text{-stirrups} = 10 \text{ in.}$$

$$s = \frac{8\,550\, d}{V} \text{ for } \tfrac{5}{8}\text{-in. round } V\text{-stirrups} = 16 \text{ in.}$$

Any one of these three sizes may be chosen, but the $\frac{1}{2}$-in. round bar seems preferable for the 30-in. beam at a minimum spacing of 10 in., which is the spacing required at the point where the first bar is bent up for shear. This spacing may be increased toward the center of the span to a point where the unit shear is only 40 lb., and the total shear is 40 $bjd = 11\,760$ lb., beyond which

no stirrups are required. This gives a distance out from each end of $\frac{33\,000 - 11\,760}{2\,200} = 10$ ft., where stirrups are required, or $30 - 2 \times 10 = 10$ ft. centrally, where no stirrups are required. The stirrup spacings for Spans 1 and 3 are determined in a similar manner.

In presenting this example sufficient descriptive matter was deemed necessary for the benefit of the reader. In actual practice, however, the complete numerical work could be reduced to about six lines for the 3-span problem. The outline and table herein given, together with the three other tables mentioned, furnish a complete equipment for the design of any concrete beam or slab in building construction.

The author's suggestion of inverting stirrups over the range of negative moments would not avail much since the stirrup ends are usually hooked over the top bars and some straight bars always extend the full length of the beam. Furthermore, the shear acts in the same direction whether over the range of positive or negative moments. The writer favors the use of stirrups merely for the purpose of supplying a deficiency in shear reinforcement wherever needed, and would utilize bent bars to the fullest extent as far as they are available in any given case.

The practical method of designing reinforced concrete beams herein given, deserves the careful consideration of structural engineers, and it is hoped that it may gain in favor to the extent of being incorporated into the building codes of other larger cities.

FINISHING AND CURING OF CONCRETE ROADS

Discussion*

By Herbert J. Gilkey, Assoc. M. Am. Soc. C. E.

Herbert J. Gilkey,† Assoc. M. Am. Soc. C. E. (by letter).‡—During the past two years the writer has conducted several carefully planned series of tests in an effort to obtain basic knowledge regarding the curing process in its relation to the weight and compressive strength of Portland cement mortars and concretes. He has, therefore, given close attention to the several recent reports§ of the California and California-Lewis Institute tests, that form the basis for that part of Mr. McKesson's paper relating to the "Curing of Concrete".‖

It would be difficult to conceive of a more auspicious setting for a series of tests. They were made at public expense and apparently, therefore, without serious handicap for lack of funds. They were conducted under experienced, and presumably expert, supervision and in conjunction with the actual work of one of the leading State Highway Departments of the United States.

In so far as the tests and numerical data are concerned it is doubtful if any agency could have turned out better controlled or more accurate work for the given conditions. This is a very valuable and timely paper because it collects as one target a group of current fallacies that urgently need correction. It is an excellent illustration of careful, costly work largely neutralized because of misinterpretation and incomplete analysis.

With the exception of that part of the discussion that pertains to Table 1,¶ what is herein said applies only to the part of the paper devoted to the "Curing of Concrete". Moreover, no consideration will be given to the data on the use of curing compounds and admixtures. No direct notice will be taken of the flexural test results, although much of what will be said applies equally to them. In general, strong concrete is strong in each of the several ways by which strength may be judged. It is of the curing process in its

* Discussion on the paper by C. L. McKesson, Assoc. M. Am. Soc. C. E., continued from December, 1926, *Proceedings*.

† Associate Prof. of Civ. Eng., Univ. of Colorado, Boulder, Colo.

‡ Received by the Secretary, October 29, 1926.

§ "Studies of Curing Concrete in a Semi-Arid Climate," by H. F. Gonnerman and C. L. McKesson, Assoc. M. Am. Soc. C. E., *Bulletin, No. 15,* August, 1925; "Curing of Concrete, Recent California Experiments," by C. L. McKesson, Assoc. M. Am. Soc. C. E., *California Highways,* January, 1926: "Curing of Concrete, Recent California Experiments," by C. L. McKesson, Assoc. M. Am. Soc. C. E., *Western Construction News,* April 25, 1926; "Studies of Curing Concrete in a Semi-Arid Climate," by C. L. McKesson, Assoc. M. Am. Soc. C. E., *Engineering News-Record,* March 18, 1926; Discussion, *Proceedings,* Am. Concrete Inst., 1926, p. 432, *Proceedings,* Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1125; *California Highways,* Vol. 2, February, 1925, p. 5; *Public Roads,* Vol. 6, March, 1925, No. 1.

‖ *Proceedings,* Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1129.

¶ *Loc. cit.,* p. 1128.

relation to compressive strength that the writer feels competent to speak with assurance.

The reported data are largely misinterpreted and the conclusions distinctly misleading. In extenuation, it is only fair to add that the same misconceptions and wrong interpretations have appeared in the published results of several other costly State highway investigational projects. They are also apparent in at least one current Government investigation of magnitude if the writer is correctly informed as to procedure followed.

In view of this, Mr. McKesson's paper is timely as a basis for discussion. In the writer's remarks, the general contentions and objections will be stated; then various points appearing in the paper will be taken up separately, after which the basis for the writer's assertions will be briefly outlined in conjunction with certain curing data. At the end a suggested procedure will be given for determining how long concrete should be wet cured.

The General Contentions.—

1.—Wet concrete has been called dense concrete and reduction in weight by drying has been mistaken for lack of density.

2.—Strengths due to drying out and those due to curing have been hopelessly confused.

3.—Relative efficiencies of curing methods have been rated on short-time strength tests (mostly 14, 21, and 28-day periods), which give more of a measure of strength due to drying than that due to curing. The few 3-month tests given are likewise on specimens all the way from the saturated to the thoroughly dried out condition at test. Factors mentioned under Contentions (1) and (2) complicate both short-time and long-time curing results but exercise a much more important relative effect on the former.

4.—It has been assumed that the concrete has been sufficiently cured when test cores show a strength of 2 000 lb. per sq. in., or better. The presumption is that 2 000-lb. concrete has then been attained. The core strengths, as found in all these tests, are the strengths of dry concrete. A soaking rain reduces the strength 30 to 40%, and, further, produces a moist sub-grade that makes the loading condition far less favorable than it was with a dry sub-grade under dry concrete. This latter condition is the logical one to use in deciding just what minimum strength should be developed before curing ceases. All exposed concrete is potentially wet concrete and no strength above that for the saturated condition can be safely assumed.

5.—Some of the curves show an unwarranted lack of agreement with the points that are supposed to define them. Other minor points will be mentioned in passing.

In Fig. 3,* the smooth curves shown are supported by relatively few and very scattered groups of points. The middle curve, especially, is pulled far from its orbit by one lone satellite. It seems far-fetched to attempt to define any regular relation for such sparse and scattered plotted evidence. In offering this criticism, no attempt is made to state whether or not the curves drawn approximate the truth. As a matter of fact they do agree in general

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1135.

trend with the writer's findings based on much more consistent evidence. The point is that for the evidence shown, a great number of widely variant curves might be drawn, some of which could make a better claim to correctness than any of those shown. The same is true of the lower curve of Fig. 1.*

The first column under "Cores" of Table 1, gives what purports to be "percentages of voids" in the cores. It is evident that these so-called voids represent approximately the volume of water that these cores would absorb. Voids and apparent specific gravities of many stones and some other materials can be found in this manner and useful constants may be obtained thereby. These materials, however, take up and lose a definite quantity of water. They can be readily dried to constant weight.

Concrete does not fall in this class of materials. Tests made at the Iowa State College,† and fully supported, in these respects, by tests by the writer, indicate that it is a rather long and tedious process to dry to constant weight. Absorption, on the other hand, is rapid, and nearly constant weight may be attained in a few hours or a day of soaking. Exposed to dry air concrete will continue to lose moisture for weeks. Placed in an oven concrete will require a long time to reach constant weight at any one temperature. Increase the temperature, and it will lose more weight. Remove it from the oven, and in even relatively dry air, it will take on appreciable weight by re-absorption of water from the air. It loses water much less rapidly after a period of moist storage than if never subjected to moist storage. At present, the writer has companion specimens in dry air (relative humidity rarely above 30% and never above 50%), some of which are taking in moisture (having been oven-dried at 70° cent. (158° Fahr.) for several weeks, but not quite to constant weight). The others have been in the air the entire period and are still losing weight.

The water in concrete is present in at least two different states. Part of it is in chemical combination and part occupies the pore space and capillary passages. The smaller passages have a strong attraction for the water and give it up reluctantly. Just when chemical separation starts the writer would not venture to say, but it is apparent that there is no abrupt change in condition as to loss of water for temperature considerably higher than the boiling point.

All these difficulties might have passed unnoticed had the procedure been one of soaking without re-drying; and this is probably what was done. The volume of water necessary to saturate the concrete was in that case a function of the dryness of the core or other specimen at time of immersion. In either case the figure obtained has little significance and is certainly not a constant for that concrete.

Talbot and Richard‡ have defined the voids in concrete as the total space in a unit volume of freshly mixed concrete, not occupied by solids (cement, sand, or stone). In like manner, the density is defined as the ratio of absolute

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1131.

† "Effects on Concrete of Immersion in Boiling Water and Oven Drying", by W. J. Schlick, M. Am. Soc. C. E., *Bulletin 59*, Eng. Experiment Station, Iowa State Coll.

‡ *Bulletin 137*, Eng. Experiment Station, Univ. of Illinois.

volume of solids to the total space occupied by the freshly mixed concrete. To use the term "voids" in the sense that it is used in Table 1, is not only misleading but betrays either an ignorance of accepted practice or a carelessness that is unjustified.

Concretes that were alike at mixing will continue to have equal voids and densities. One may dry to a weight of 142 lb. per cu. ft. while the other is saturated and weighs 146 lb. per cu. ft. In the former, part of the water voids have become air voids. The absolute volumes of cement, sand, and stone remain unchanged, however.

What Table 1 apparently shows is that the less dense the concrete, the stronger it is. This is absurd of course. What it does show is that a dry core is stronger per unit area but lighter per unit volume than a saturated core or cylinder of the same concrete. This is true. The conditions would have been reversed, however, were the cylinder dried and the core soaked prior to test. The one standard condition is to have both specimens saturated at test since it is difficult to attain equal degrees of dryness. In comparing cores with standard laboratory cured cylinders (wet cured, tested wet), the core should always be immersed for about one day prior to the test. Strength results are then quite comparable and weight results reasonably so.*

The actual densities of these concretes were probably 0.80 to 0.82, that is, from 20 to 18% voids, instead of the 9 or 10% given. These figures are reached by reconstructing the mixture in accordance with the incomplete data given in the report of parts of these curing tests.† The tests here reported doubtless used materials and proportions not greatly different, since the aim throughout seems to have been to approximate usual field conditions.

Data for each batch were as follows:‡ Cement, 156 lb.; dry sand, 378 lb.; dry gravel, 522 lb.; water, 87; weight of wet concrete, 152 lb. per cu. ft.; and water-cement ratio, 0.84. Normally, the concrete was to contain 6 bags of cement per cubic yard in place.

The volume of water per cubic yard of concrete was then $0.84 \times 6 = 5.04$ cu. ft.; water voids $= \dfrac{5.04}{27} = 0.186$. The total voids consist of all the water in the mixture plus any air voids. It is seen that the water voids alone are twice the total volume of voids as given by Table 1. For a mixture as wet as this, air voids may be assumed at 0.01, which gives a total of 0.196, or a density of about 0.80.

A careful analysis made by assuming reasonable values for the specific gravities of aggregates by trial until a concrete weighing 152 lb. per cu. ft. was obtained, gives results nearly identical with the figures just given.§

* Concrete that has once dried out will not re-absorb quite as much water as if it had been kept continuously wet (after one day or so). This is probably due to entrapped air that has replaced the evaporated water and that is not subsequently dislodged.

† *Bulletin No. 15,* Structural Materials Laboratory, Lewis Inst., Chicago, Ill.

‡ *Loc. cit.* p. 6.

§ In *Proceedings,* Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1131, it is stated that the water used in these tests was about 8% of the total weight of the cement, sand, and gravel. The figures used from *Bulletin No. 15,* of the Structural Materials Laboratory, Lewis Inst., would make the water $= \dfrac{87}{156 + 378 + 522} = 8.2\%$, which agrees closely. The concrete described at the beginning of Table 2 (*Proceedings,* Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1134), was of essentially the same proportions, having 6 bags of cement per yd.

If the specimens of Table 1 were originally from the same or similar concrete, their densities and, therefore, their voids (voids = 1 — density) were identical, or nearly so, as made. Evaporation or absorption does not alter the status. Voids consist of air plus water. Evaporation will lower the weight but it does not alter the voids. Water voids simply become air voids. A re-soaking returns air voids to water voids. Weights vary, densities and voids remain constant.

What in the paper may appear to be a more or less incidental detail or a slip of the pen, has been discussed at some length because it is the kind of incidental detail that leads to very serious misconceptions. Moreover, the author is by no means the only offender in this regard. There is a very general tendency to compare indiscriminately field concrete specimens of varying degrees of dryness with strengths and weights of wet tested laboratory concrete. This may be actual or by inference. Even if there are no actual laboratory specimens, the field specimens show up extra well because of dryness at test, and every one is pleased. A Government efficiency project of considerable magnitude appears to be at the present time staking its results on this same fallacy.

Dry concrete is, however, always potentially wet concrete in a pavement or other exposed construction. An hour or two of soaking rain and the extra strength is gone. Moreover, with a wet sub-grade the condition is far worse than when the sub-grade as well as pavement is dry. Similar statements were made earlier in this discussion. They cannot be too often repeated or too strongly emphasized.

The one point of value in Table 1 is from the data in the column entitled "Strength Ratio", which gives an excellent comparison of relative strengths of similar concretes dry at test *versus* wet. Moreover, these values are exactly representative of the true difference as will be shown by the plotted test results (Figs. 7 to 13, inclusive).

The criterion for length of moist curing is given by the author as the period necessary for the concrete to develop sufficient strength to sustain traffic.* This is entirely sound except that Mr. McKesson's standard of strength is that of dry concrete on a dry sub-grade. When once dried out concrete ceases to gain further strength. The curing will resume, as Mr. McKesson has stated, when the rains come. This resumed curing is, however, at a very much slower rate than the original.† There will be a considerable period, therefore, when the strength of the pavement is only that of a saturated concrete on a moist sub-grade. This is the critical period in the life of the pavement and this is the condition that should be the criterion for the curing period and method.

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1130.

† For example, specimens immersed in water after 28 days in dry air had at the end of 3 months the same strength as similar specimens moist cured for 28 days only. Later tests verify these figures. For further illustration note the "In @ 25" and "In @ 61 day" curves on Fig. 7. The strength drops from the *C*-curve to the *D*-curve (about 20% of the 28-day standard strength of 25 to 30% of the *C*-curve strength) at once, due to wetting. It then gradually rises by resumed curing until about 1 month later and 5 weeks later, respectively, it has attained in the wet condition, the former dry strength. Strengths equal to the 28-day standard are reached at ages of about 65 and 130 days, respectively, *Proceedings*, Am. Concrete Inst., 1926, Fig. 13, p. 424.

It is not here urged that the period of moist curing be unduly prolonged. Detours and curing are costly. It is not for the writer to say what is justified in a particular case. His sole contention is that correct data should be used in reaching a decision. In some cases it might well be advisable to increase the richness of the mixture in order to decrease the curing period. In other instances traffic might be turned directly on the moist earth covering or adequate sprinkling maintained under traffic. These points will be considered subsequently in detail.

The references to water necessary for given length or periods of curing* indicate a somewhat peculiar point of view. There is little doubt that any workable concrete mixture contains more than enough water to satisfy all curing needs for any length of time. The only reason that watering is necessary, is the practical impossibility of conserving the mixing water. The specimen dries out. Wet-mixed concrete dries out as badly and even worse than stiff-mixed concrete. It is less dense and loses water with greater ease. There is no advantage in using extra mixing water with the idea of helping the curing process.†

The writer's remarks might by implication lead one to the fallacious expedient of using excess water in mixing for the purpose of prolonging the curing period. Advocates of wet concrete die hard and are only too ready to grab at such a fine fat straw.

Mr. McKesson is correct in his statement‡ that the loss of water is much less, in a given time, for concrete that has had a few days of moist curing, than for a concrete exposed at once to the air.

The data of Fig. 1, lower curve, throw considerable light on different curing methods as regards their efficiency in keeping the concrete moist. The points tell little as regards relative strengths. The lower the location of a given point, the more complete the drying out. The bottom points represent specimens that are quite dry and in which curing was halted early. The right-hand points (strongest specimens) are only about half dried out. The moist earth covering although only wet for 3, 7, or 11 days, was effective in keeping the concrete moist for a long time after sprinkling ceased. These specimens (Points 7, 11, and 13) are therefore reasonably strong because of relatively long-time curing and also strong because of partial drying. On the other hand the upper points (Points 32 and 12) have dried out least.

Potentially, these are by far the strongest specimens but the fact that they are wettest at test makes them show up less favorably. Thus, Fig. 1 shows strength comparisons between wet concrete at the top and very dry concrete at the bottom with many gradations between. Were these specimens all made comparable at test by immersion in water prior to test, the top point (No. 32) would remain fixed. All others would move up to the level of Point 32 and to the left distances more or less proportional to their present distance from the top. The final position would show Point 32 at the extreme right with the other points ranging to the left at about the same level,

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, pp. 1130–1131.
† *Proceedings*, Am. Concrete Inst., 1926, Table 11, p. 408; Fig. 6, p. 415.
‡ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1132.

and in a more or less continuous alignment. There would be no abrupt bend in the curve. The 100% ordinate would then be that of Point 32. All points near the bottom would move much farther to the left than their present location since there would be present both the weakening effect of very brief curing and also a great weakening due to being wet instead of dry at test. They would move up by virtue of the added weight from soaking.

On the other hand the bottom point (No. 1) could be made the pivot by bringing all other specimens to the same approximate degree of dryness. Then the migration would be to the right and the relative arrangement identical with that just described, except for elevation. Point 32 would continue to be the right-hand point of a more or less horizontal line at the lower level.

The author mentions the possible effect of wetting on temperature.* The temperature of the water applied was not greatly different from that of the concrete. After duly considering the possibility of some cooling action from evaporation, after the manner of the wet bulb thermometer or South African water bag, it is scarcely probable that temperature changes due to wetting by any method whatsoever had any measurable effect on curing within the range of temperatures that existed (about 45° to 110° Fahr. from Fig. 5).† A very slight shortage of water would be far more deleterious than a change of many degrees in temperature. Moreover, curing halts when a certain degree of dryness is attained. No amount of favorable temperature will produce a further gain in strength as long as moisture is lacking. The matter of temperatures and their relation to rapidity of curing is, however, a question that needs experimental treatment. The trend is known but definite numerical relationships are undefined. The writer's comment on this phase represents only his own opinion, as opposed to the similarly unsupported statement made by the author.

Some of the author's data and deductions‡ again illustrate the oft-recurring confusion due to the extra strength of dried out concrete compared with concrete better cured, perhaps, but wet at test. The concretes of Table 2§ represent highly indeterminate conditions, especially difficult to evaluate in time tests as short as 28 days and less. All specimens were kept covered with earth for 14 days, but the watering period was varied. The earth undoubtedly retained moisture sufficient to continue curing for several days after watering ceased. Lack of drying out produced strength by added curing. Drying out reduced that strength by halting the curing, but added that due solely to drying. Without relative weight data, it is impossible to state to what extent a given strength was due to one cause or the other. The apparent superiority over the wet-cured, wet-tested standard laboratory specimen is, however, due in all cases to the partial drying of the field specimen. Immersion of the cores a day before the test would have shown a very different set of ratios. These would give information of value.

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1133.

† *Loc. cit.*, p. 1136.

‡ *Loc. cit.*, pp. 1134–1135.

§ *Loc. cit.*, p. 1134.

Fig. 3 illustrates the fact that short-time strengths are practically a straight-line function of the period of moist curing to within a few days of test. These few days represent the time required for concrete to dry out appreciably in the particular climate for that particular surface volume relation. Fig. 3 shows that this period for these tests was from 11 to 16 days.

Fig. 4* shows that the drying out was much slower in the Illinois tests cited than in the California tests, which is to be expected. The usual confusion between wet and dry concrete seems to exist.

It is interesting to note in Fig. 5 that relative humidity was at times more than 90 per cent. There is no question but that concrete rather dried out and thirsty would absorb quite a little atmospheric moisture at these higher humidities. The numerical evaluation is not at present possible and it may or may not have been appreciable in its effect on strengths.

The fact that air-cured concrete always showed a low surface hardness† is to be expected. It is low-strength concrete. Moreover, the Lewis Institute report of the tests shows the highest values of surface hardness to be for the concretes having the longest periods of moist curing. This property is less influenced by the "wet-at-test" condition, for in a very short time the surface dries out sufficiently to acquire the hardness of dry concrete.

Data from Tests by the Writer.—The writer has spoken with positiveness regarding curing and its relation to compressive strength. In this territory he is certain of his ground and the general correctness of his contentions.

Complete curing series have been run on 2 by 4-in. mortar specimens and 6 by 12-in. concrete specimens. An outline control series has been run on 6 by 12-in. mortar specimens. In addition, several partial series have been conducted on several different concretes and mortars to check and verify certain phases of the problem. Some of these results are already a matter published of record.‡

A sufficient amount of evidence from a later and more extensive series that corroborate the preceding findings will be incorporated with this discussion to drive home, if possible, the fact that short-time tests give no fair measure of merit of the relative curing methods and that it is not valid to compare the strengths of concretes dry at test with those wet-tested as measures of the relative excellence of the concretes in question.

In Fig. 7, the *A*, *B*, *C*, and *D*-curves show a "universal" curing diagram for 1:2:3 concrete having a water-cement ratio of 0.90. The specimens were 6 by 12 in.

In Fig. 8, the *A*, *B*, *C*, and *D*-curves show the four main lines of Fig. 7. The *A′*, *B′*, *C′*, and *D′*-curves show four lines for 6 by 12-in. mortar specimens exactly like the concrete specimens except for the omission of coarse aggregate, and the *A″*, *B″*, *C,″* and *D″*-curves show the diagram for 2 by

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1136.

† *Bulletin No. 15*, Structural Materials Laboratory, Lewis Inst., Fig. 7.

‡ "Curing Conditions of Mortars and Concretes," H. J. Gilkey, *Proceedings*, Am. Concrete Inst., 1926, pp. 395–436; "Autogenous Healing of Mortars and Concretes," H. J. Gilkey, *Proceedings*, Am. Soc. for Testing Materials, 1926.

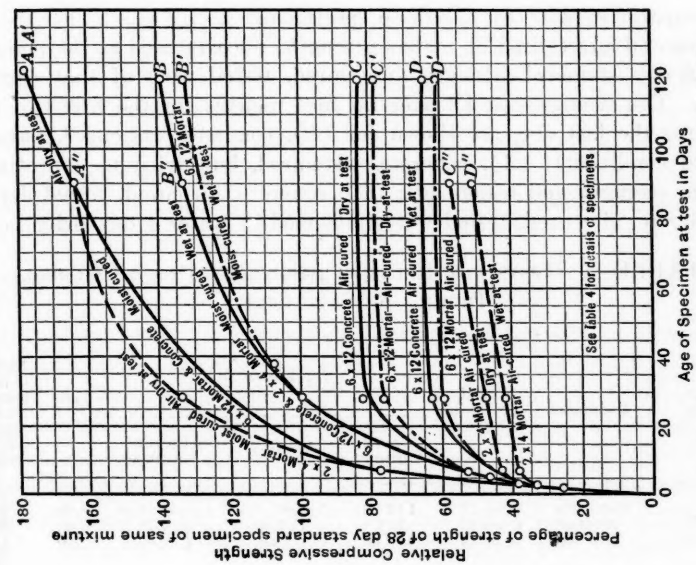

FIG. 8.—ABRIDGED UNIVERSAL CURING DIAGRAM FOR MORTARS AND CONCRETE.

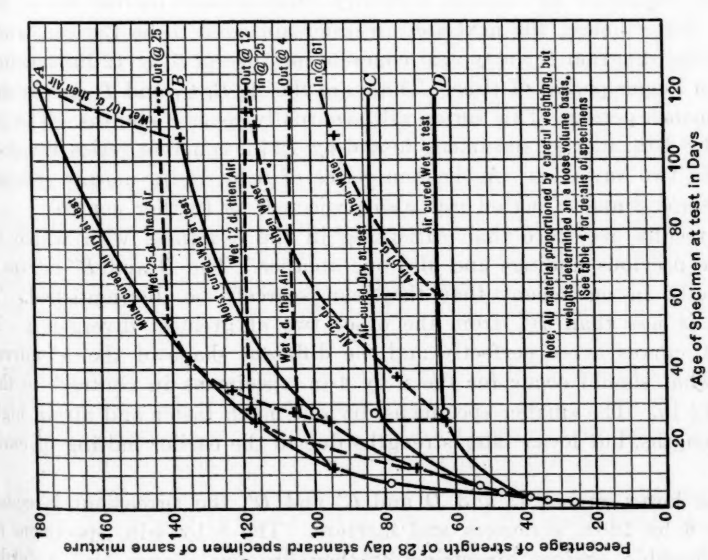

FIG. 7.—UNIVERSAL CURING DIAGRAM.

4-in. mortar specimens of a different mix $\left(1:2.26;\ \frac{w}{c} = 0.70\right)$. Table 4 gives comparative data for all these specimens.

The several curves in Fig. 8 show strengths all expressed as the percentage of the 28-day standard strength (wet-cured, tested wet) of that particular mixture. The strengths at all ages up to 4 months for the first two, and 3 months for the last one, are shown for (*B*), concrete, wet-cured, tested wet (standard condition); (*A*), concrete, wet-cured, but dried to approximately maximum strength and tested dry; (*C*), concrete, air-cured, tested dry; and (*D*), concrete, air-cured, tested wet (immersed in water 1 day prior to test).

TABLE 4.—TESTS FOR RELATIVE STRENGTHS DUE TO CURING. DETAILS OF SPECIMENS.

Curves.	Materials.	Size of specimen, in inches.	Proportions.			$\frac{w}{c}$.	Strength, in pounds per square inch, of standard 28-day specimen.
			Loose volume.	Weight.	Absolute volume.		
A, *B*, *C*, *D*........	Concrete	6 by 12	1 : 2 : 3	1 : 2.06 : 2.52	1 : 2.48 : 3.18	0.90	2 950
A', *B'*, *C'*. *D'*.....	Mortar	6 by 12	1 : 2	1 : 2.06	1 : 2.48	0.90	3 600
A'', *B''*, *C''*, *D''*....	Mortar	2 by 4	1 : 2.26	1 : 2.5	1 : 2.96	0.70	4 200

The *A''*, *B*,'' *C''*, and *D''*-curves have been published.* The general trend was fully supported by various auxiliary concrete and mortar series as reported. Nevertheless, the next step in order appeared to be the construction of a similar diagram for a typical concrete mixture of 6 by 12-in. specimens, and for a longer period of time. These are the *A*, *B*, *C*, and *D*-curves shown for a 4-month period. This series will eventually cover 8 months. The jump from 2 by 4-in. mortar specimens to a 6 by 12-in. concrete series involved a change in two variables. A skeleton series of 6 by 12-in. mortar specimens for the same 8-month period was also projected, as further control.

The results are more than satisfying in the exactness with which they check the previous findings and also one another. The *B* and *B''*-curves are so perfectly in agreement that they can scarcely be distinguished. The *B'*-curve is less than 7% from the other two at greatest divergence. The *A* and *A'*-curves agree perfectly and the different shape of the *A''*-curve is exactly what should occur for the 2 by 4-in. specimens in contrast to those of 6 by 12 in. The smaller specimens dry out much faster and attain higher early strengths, but lower later strengths due to the earlier halting of curing action.

In the lower curves, *C* and *D* and *C'* and *D'*, the agreement is equally good for 6 by 12-in. concretes and mortars. The 2 by 4-in. specimens are more vulnerable against air-curing as they dry out sooner and a further gain in strength practically ceases. Just why the *C''* and *D''*-curves are less flat than the other similar cases is the one factor not explainable.

* *Proceedings*, Am. Concrete Inst., 1926, Fig. 13, p. 424.

These several series justify the following general conclusions:

1.—Mortar and concrete specimens of similar proportions and sizes respond to curing conditions in identical manners and in essentially equal degrees.

2.—Specimens of different size respond to moist curing in exactly the same manner and extent. For other treatments, involving drying out either before or after moist storage, their relative strengths are effected by the variation in the rate of drying.

3.—Specimens of the same size subjected to environments different in drying properties would be affected in exactly the same relative manner as specimens of different sizes subjected to the same atmospheric environment.

Fig. 7 fully supports the following additional conclusions previously enunciated:*

4.—Concrete cured wet will, on removal from water, take on at a relatively rapid rate considerable additional strength, from 20 to 45% in these tests. The time to attain maximum strength is a function of the relative dryness of the atmosphere, the relation of exposed surface of specimen to its volume, etc.

5.—When concrete has reached a certain degree of dryness, further gain in strength with increased age practically ceases. The strength remains at a standstill as long as the concrete is kept dry. This means that the relative virtue of dry concrete in comparison with concrete that is continually wet diminishes with age as the wet concrete takes on added strength along Curve *B* for an indefinite period. The dry concrete is stationary; that is, the curve is horizontal, or practically so.

6.—A short-time soaking of any concrete of dry strength along either Curve *A* or Curve *C* will at once lower the strength to that of wet concrete along Curve *B* or Curve *D*. This has been duly verified by test. The phenomenon is entirely reversible. Strengths can be lowered or raised at will by soaking or drying. After each soaking there is, of course, some increase in strength due to resumed curing in the moist interval.

Figs. 9 to 13, inclusive, are derived from the universal diagrams (Figs. 7 and 8). When once the diagram is properly constructed and adequately controlled, any real or assumed curing condition may be investigated. The figures for "Apparent Relative Strength" (broken lines) represent, in all cases, the ratio of the strength as shown along the "out" curve (Fig. 7) to the corresponding ordinate of Curve *B*. This is merely an apparent strength since all strengths along Curve *B* are wet strengths. The "out"-curve strengths are all dry strengths after the first few days subsequent to removal from the water. Specimens remained somewhat moist for a few days.

The "Actual Relative Strengths" (solid lines) are the "out"-curve ordinates (Fig. 7) divided by the *A*-curve ordinates. In other words the ratio of dry strength to dry strength may be used. The same result could be obtained by immersing the "out" specimen a day before test and taking the ratio of this strength to the standard wet strength shown along the *B*-curve.

* "Curing Conditions of Mortars and Concretes," H. J. Gilkey, *Proceedings*, Am. Concrete Inst., 1926, pp. 395–436.

Fig. 9 shows the relative and actual strengths for 6 by 12-in. concrete and 2 by 4-in. mortar specimens removed from the water and exposed to the air after 4 days of age (3 days in water). Figs. 10 and 11 give the same information for longer periods of moist storage, namely, 12 and 25 days, respectively. All the curves give these data for both series, *A, B, C* and *D* and *A″, B″, C″* and *D″*. In other words the two similar sets of data should be an aid in evaluating the effect of varied rates of drying out.

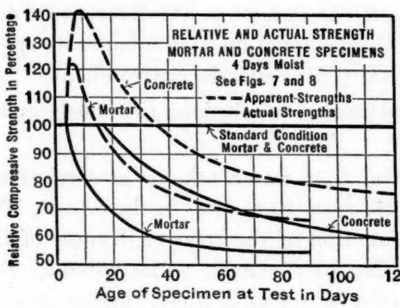

Fig. 9.

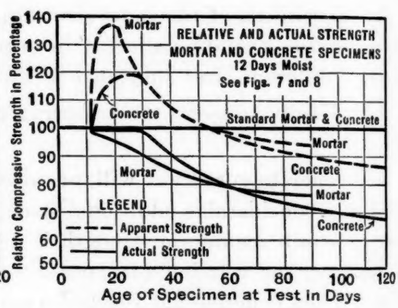

Fig. 10.

The upper part of Fig. 12 shows the apparent strength of dry concrete in terms of wet, and on the lower part the apparent strength of wet (standard-cured) concrete is shown in terms of that wet-cured but dry at test. These

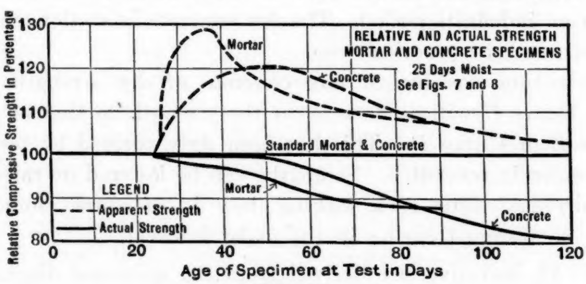

Fig. 11.

are all apparent strengths. Actual strength ratios are all along the 100% line. The dry concrete above, if wet, would fall to this line, whereas the wet concrete below, if dried, would rise to it.

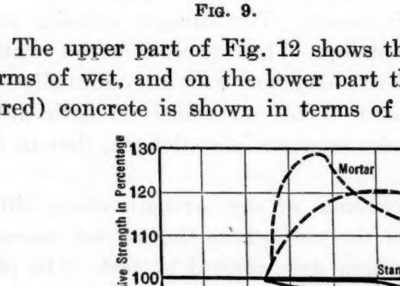

Fig. 12.

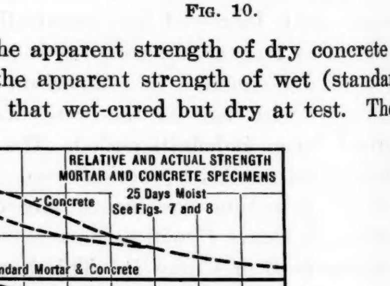

Fig. 13.

Fig. 13 affords a check on some of the previous assertions. If the ratio of dry strengths to dry strengths is identical with that of wet strengths to wet strengths, then $C : A :: D : B$. (This of course refers to the respective ordinates of the curves.) These two ratios show remarkable agreement for both the concrete and the mortar.

It will be noted in Fig. 9 that concrete out at 4 days gives apparent relative efficiencies as follows:

Test age (days)................	7	14	21	28	60	120
Apparent relative strength (percentage of standard specimens wet at test).................	141	136	116	105	87	76
Actual relative strength percentage of standard specimens dry at test	100	100	94	88	71	59

The actual strength ratio of 100% at 7 and 14 days simply indicates that until that time the specimen had not yet dried out sufficiently to halt curing.

Notice that the trends of both actual and apparent strengths are parallel and downward. Had longer time tests been conducted in the California investigation, the fallacy of basing conclusions on comparisons at ages of 28 days or less would have appeared in spite of the fact that strengths of wet and dry concrete were indiscriminately compared.

It will be noted in Fig. 11 that even 25 days of moist curing shows an efficiency of only 80% that of continuous standard moist curing at the end of 4 months. It is apparently 120% efficient at 50 days. Moreover, the small mortar specimens, through early drying, because of difference in size, seem to be at a disadvantage at first. Within the time limit of these tests the concrete has in several instances already overtaken the mortar in its downward trend.

These curves will bear careful study. It is hoped that they will aid in sufficiently clarifying the subject of curing to prevent a repetition of the four major fallacies so evident in the interpretation of these tests, namely:

1.—Indiscriminate comparisons of strength of dry and wet concretes as criteria of relative curing, or any other, efficiencies.

2.—Basing conclusions as to relative efficiencies of curing methods on the results of short-time tests which fail to distinguish adequately between the curing and drying that are both present.

3.—Confusion of wetness with density.

4.—Assuming the strength of dry concrete supported on a dry sub-grade as a proper basis for deciding when curing may be safely halted.

Conclusions and Recommendations.—If the handling of Mr. McKesson's data be as seriously at fault as claimed, then it must follow that his conclusions are probably of little value. What then shall be substituted for the three simple recommendations * that appear to be such a happy and usable ending?

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1138.

First, it must be recognized that the most severe test in the life of the pavement will not necessarily come with the first turning on of the traffic. Nor is there much question relative to an old pavement.

When traffic is turned on, the concrete may be quite dried out and the sub-base firm. On the other hand old concrete will have passed through several alternating periods of dryness and wetness. Curing halts during dry periods but resumes at a slow but appreciable rate during wet periods. Thus, any incomplete early curing is remedied in the course of time and the concrete gets better and better with age. Even a little moisture retained in the sub-grade is a great aid to the concrete in so far as curing is concerned. The real test comes when the concrete gets its first rewetting on a softened sub-grade. The resumed curing is at altogether too slow a rate to help materially in this particular emergency.

First, then, a decision must be made as to what strength will be needed to tide the pavement over this period. That decided, this strength should be expressed as a percentage of the 28-day standard strength of the concrete being used (this means the wet-cured, wet-tested, 28-day strength). Then reference to Curve B (Fig. 7 or Fig. 8) will indicate what period of wet curing is necessary to develop about that percentage of the 28-day standard strength.

During reasonably dry weather, traffic may be turned on before curing is complete. The sub-base is firm even if the concrete be kept moist for curing. The curing should be kept up, simultaneously with use, however, in preparation for that more critical period when both pavement and sub-grade are wet. For example, if 2 500-lb. concrete is being used and it is decided that it should have a strength of 2 000 lb. before being placed in service, it is found that about 80% of the 28-day standard strength will be developed in 16 days of moist curing.

Along Curve A, which shows both the hardening and curing, the 80% strength is attained when the concrete is about 8 days old. The concrete, if still being cured, is wet, but the sub-grade, in good weather, should be firm and the worst condition is not present. Therefore, traffic may safely be turned on somewhere between 8 and 16 days. However, in preparation for the evil day to come, when the concrete will be wet, and the sub-grade soft, precaution must be taken to insure the equivalent of a full 16-day moist curing period.

This does not necessarily mean that the pavement must be kept ponded or covered for the full 16 days. In a truly dry climate this should doubtless be approximated. In climates less dry there might be sufficient residual moisture from 10, or even 7, days of actual soaking to enable curing to proceed without serious interruption through the full 16-day period. If the sub-grade be even reasonably moist, quite a little reliance can be placed on the moisture that it will furnish through capillarity. During dry months when the foundation is firm, traffic can be turned directly on the moist earth covering and the pavement can be used while it is also being strengthened against the future test. If a moist earth covering is not used, added strength can be attained by adequate sprinkling while in service. In a windy or dry

climate too much reliance, however, should not be placed on the sprinkling, as much of the water is lost in a short time by evaporation. In many cases it would undoubtedly be excellent economy to use a richer mixture in order to reduce "out-of-service", detour, and curing costs.

There are two or three points in which the element of judgment must enter to an objectionable degree:

(*a*) Just what strength will be sufficient to tide over the emergency period is a problem involving the nature of the sub-grade and the still unsolved problem of how a yielding sub-grade supports a rigid pavement under more or less concentrated loading. That problem is no more an inherent part of the situation than it is to the whole matter of suitable design stresses, pavement loading, etc. This is still a question for the investigator and expert analyst.

(*b*) How long curing will continue after direct application of water has ceased, is a real problem for the field man. By careful and intelligent observation he should be able to judge this quite satisfactorily. A moist earth covering left on is one of the most effective methods for retention of moisture.

(*c*) Curve *B* may not be indicative of the time rate of gain in strength for all concretes, but it is typical and doubtless closely approximates most cases of standard curing. A better curve of course could be obtained from a few specimens of the concrete in question.

Barring poor construction, including the placing of concrete pavements on loose fill, insufficient allowance for expansion and contraction, etc., it is probable that initial or incipient failure usually dates to the critical period when relatively young saturated concrete is being called on to support loads on a soft sub-base.

Late fall pavement, with retarded winter curing because of low temperature, is very likely to suffer serious damage during the spring thaws. This case is not of interest in California, however.

Acknowledgment.—The writer is indebted to William H. Thoman, Instructor at the University of Colorado, for help in preparing the data.

AERIAL SURVEYS FOR CITY PLANNING

Discussion*

By Theron M. Ripley, M. Am. Soc. C. E.

Theron M. Ripley,† M. Am. Soc. C. E. (by letter).‡—The aerial survey and map which is being made of Erie County, New York, is primarily for the purpose of the general study of the Greater Motorway System and preliminary ground location of the various roads comprising that System. The survey was authorized by the Board of Supervisors and ordered by the County Engineer, George C. Diehl, M. Am. Soc. C. E.

As the first order for this work covers the City of Lackawanna, extending a short distance into the City of Buffalo, and, later, is to cover the entire city, it may be apropos to discuss Mr. Matthes' paper, at least in part, from experience gained in this work.

To one desiring to secure an aerial survey of his city or county the writer would say "do not be discouraged if your officials do not immediately accept your recommendations and make the appropriation to carry them out". It will be necessary to "sell" them the proposition, unless by rare good fortune they are already "sold", as the survey will seem (to them) of far less value than it is; moreover, the amount of money requested will look much larger in a lump sum, than if the same or a larger amount was paid out in monthly pay-rolls. The "selling" campaign should be kept up, however, because in no other way can the same amount of detailed information be acquired in so accurate a manner.

When one mentions a cost of $100 per sq. mile it does not sound very large, but as soon as it is multiplied by 200 or 1 000, the impression is entirely different. The latter is exactly what engineers must do if they enter on a large city or county proposition and desire to have the plans as their exclusive property.

Aerial surveys have been made of large areas and the ownership of the negatives left in the possession of the company that did the work, the client thereby receiving his maps at a greatly reduced price. In Erie County this is not believed to be good practice. Therefore, the work is the exclusive property of the County. Any engineer or board working on a general planning scheme will easily realize how his or its work can be hampered by conflicting opinions arising from a general distribution of the survey maps before plans are perfected and, therefore, the great advantage of curtailing such distribution. Such conflict could easily cost the city or county many times the difference in cost between private and general distribution of maps.

By clientele ownership, as outlined, the writer does not mean that any one but the board of officials directly responsible can have access to the

* Discussion on the paper by Gerard H. Matthes, M. Am. Soc. C. E., continued from December, 1926, *Proceedings*.

† Chf. Engr., Greater Motorway System, Erie County, Buffalo, N. Y.

‡ Received by the Secretary, November 29, 1926.

maps; in fact, the officials of the Greater Motorway System are proud of their aerial survey and map and are only too glad to show it to any one and furnish any information relative thereto, but they must come to the office for such examination.

The aerial work of this survey was done by the local firm of Ronne and Washburn and all the ground work, from developing the negatives to the finished map, by the Fairchild Aerial Surveys, Inc., of New York City. The flying was done at an altitude of 9 600 ft. and the finished map is 600 ft. to 1 in.

The work is a controlled mosaic and, with few exceptions, control points are 2 miles or less distant from each other. The traverses of existing roads were used for the control. Most of these roads have been surveyed for existing highway improvements and the notes of these surveys were used in plotting the control; nevertheless, it was necessary to run 40 miles of new traverses. This field work and the plotting of the total control of 121 miles cost $1 100, which includes only the cost of paper, tracing cloth, and the salaries of the men actually employed on the field and office work; in other words, no overhead of any kind.

Had it been necessary to traverse the entire 121 miles it is estimated this cost would have been at least $2 100. If this work had been done by inexperienced men, the cost could easily have been 50% greater. As stated by Mr. Matthes, too much time and money should not be spent on undue refinement for accuracy of control. This accuracy should depend on that desired in the completed map. The platting of a control map in the ordinary drafting room, when such map is to cover 170 sq. miles of territory on a scale of 8.8 in. to the mile, is quite a task. The sheets are large and difficult to handle and to retrace the work for the purpose of closing an error in traverse may be time and labor wasted.

In attempting to close traverses several miles in length it is readily seen that a failure to close when the error is less than 1% is, for all practical purposes, accuracy. One of the traverses on the work described, 13 miles in length, showed just such a difference with the result that a closure was effected by "fudging", namely, swinging the tracing about a selected point of the original platting and thereby saving several hours of work.

The map was made on the basis of a variation of not to exceed 1% between the control map and the aerial map, with points on the former not more than 2 miles apart. In other words, the aerial map had to scale the same as the control map up to 8.8 in., with a variation not to exceed 0.088 in., that is, scaling 1 mile each way from control points 2 miles apart.

Seven courses taken at random from the first map received and varying in length from 4 960 to 11 640 ft. on the control map, showed a maximum variation of 1.55% on a course 5 780 ft. long and a minimum variation of zero on a course 4 960 ft. long. The sum of the several courses as scaled on the control map equalled 52 410 ft. and on the aerial map, 52 490 ft., that is, the errors were both plus and minus.

For the purpose of general study and planning it is at once apparent how trivial 1% or even 2% in variation of map distance becomes when the error

is not cumulative and when such error does not exceed 0.1 in., or even 0.2 in., in open country. Where the control points can be closer together, as in street areas, this error can be reduced to 0.01 in. on a 600-ft. block, closer than the draftsman can plot it.

The map of Erie County is being furnished in atlas sheets covering an area of 2½ miles east and west and 1½ miles north and south between border lines. This produces sheets 17 by 27½ in., including the binder edge. The work is also protected in maps covering about 60 sq. miles each. The former is for use in the field and office and the latter for office use only.

The ability to lay out preliminary lines is only one function of an aerial map; in fact, in country similar to that around Buffalo the ability to show a property owner where the line is located, and why it is so located, is of more value than the mere location of the line. For these reasons the securing of rights of way becomes a prominent function of these maps.

During the past summer (1926) a serious accident happened near one of the town parks, and a suit for damages was brought against the Town Board. Testimony was given by both sides that the accident happened at or adjacent to a certain depression. The County Engineer had furnished the town officials with an enlarged aerial map of the locality in question. This map was entered as evidence, the depression was located thereon, and shown to be off the property owned by the town. The plaintiff lost.

Several statements or suggestions should be emphasized to those about to secure aerial surveys. As Mr. Matthes states, an aerial survey, or general plans therefrom, cannot take the place of ground surveys and detailed plans for construction and pay-estimate purposes. Further, "controls" need be no more accurate than the purpose for which the map is to be used and money should not be spent to secure a greater accuracy.

An aerial map is a picture to scale of all the camera sees; and the limit of its usefulness is only determined by the activities of the locality mapped and the people using it.

Obliques.—For purposes of general discussion before boards and commissions oblique aerial photographs are frequently of more value than an aerial map. This applies with equal force when showing the general layout of a property, such as a subdivision, with any included structures as buildings, bridges, etc.

The oblique is a photograph only and must not be confused with the map; because it is a photograph and shows the terrain in perspective, as is the case in all pictures, the general observer is able to get a clearer idea of the relative locations of points than when they are shown to him only on a map.

Moreover, the low altitude at which obliques are usually taken permits of showing in greater detail matters which are obscured in the higher flights. Also, the effect of presenting a beautiful photograph for the consideration of non-technical men who are passing on a proposition should not be overlooked, and an oblique is just such a picture.

PRODUCING CONCRETE OF UNIFORM QUALITY

Discussion*

By Messrs. Harry C. Boyden, and William H. Adams

Harry C. Boyden,† M. Am. Soc. C. E.—The author has stated that scientific control costs money. Is it not true, however, that proper supervision placed on the work—without any attempt to enter into scientific control—would have cost exactly the same? In other words, the institution of scientific methods on the hydro-electric work did not entail added cost, above that of proper supervision for any good job.

Another question: In placing 650 000 cu. yd. of concrete on these projects what has been the tangible and intangible saving in cement beyond the amount that would have been used by following ordinary methods? Apparently, savings have been effected on that work running into hundreds of thousand of dollars, but the author is altogether too modest to claim the credit. This saving was due to the methods of control used. These methods did not entail additional expense and, more important than the saving in dollars, they obtained the quality of concrete called for in the design of the structures.

William H. Adams,‡ M. Am. Soc. C. E.—The author has discussed a type of work which comes under strong centralized control. It would be interesting to hear discussion of possible control of quality of concrete on a more commercial job.

In the ordinary commercial building operation the structural plans are frequently made for the architect by a consulting structural engineer. The inspection and supervision of all work, including structural, in the majority of such cases are under the control of the architect's superintendent who is rarely an engineer, although usually well versed in building construction.

On by no means all such operations is the cement tested by an independent laboratory, although such test requirements are usual in the most important work. Has a point been reached where the control of the physical quality of aggregates can be delegated to a standard commercial testing laboratory? Are laboratories equipped for this service?

With such marked improvement in strength of resultant concrete as Mr. Young's paper indicates will follow from the use of carefully controlled water-cement ratios, it should not be difficult to make an owner understand that efficient laboratory inspection would be an investment and not an expense.

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* Discussion on the paper by Roderick B. Young, Esq., continued from November, 1926, *Proceedings*.

† Berkeley, Calif.

‡ Cons. Engr., Detroit, Mich.

WATER-RATIO SPECIFICATION FOR CONCRETE

Discussion*

By Messrs. Harry C. Boyden and L. A. Perry

Harry C. Boyden,† M. Am. Soc. C. E.—The tests mentioned in the paper invariably show higher results than the strengths designed for. Is it not true that the curve, $S = \frac{14\,000}{9^x}$, was used in designing the mixture rather than $S = \frac{14\,000}{7^x}$?

There has been considerable confusion in this matter. The original curve published by Professor Abrams was $S = \frac{14\,000}{7^x}$, which indicated the average strengths obtained with varying values for $\frac{w}{c}$ (water-cement ratio). In later publications the curve, $S = \frac{14\,000}{9^x}$, has been used, which gives the minimum strengths obtained with varying values for $\frac{w}{c}$. When this curve is used in designing mixtures the actual results nearly always average higher than are called for by the design and this has caused confusion and comment.

A word in defense of the slump test may not be out of place as the authors have rather discouraged its use. Although this is not a perfect test of strength, nevertheless it has its place in controlling the quality of concrete. It has been demonstrated many times that it will serve as a check against the use of varying quantities of water in successive batches. It is possible on almost any job of concreting to improve the quality by using the slump test. The procedure is to establish the consistency desired or needed for that particular job, to find the slump, and then to keep all batches at the same slump. This will give a much better control of the quantity of water used than is found on the average job.

There is a record of a famous dam where the quality of the concrete was more than doubled by a much cruder method than the slump test. During the first year of construction the only control of consistency was by keeping the concrete stiff enough to prevent a man from sinking into it over the tops of his rubber boots. Then the rule was changed and orders issued that he must not sink in more than 10 in. Immediately the compressive strength

* Discussion on the paper by F. R. McMillan, M. Am. Soc. C. E., and Stanton Walker, Assoc. M. Am. Soc. C. E., continued from December, 1926, *Proceedings*.

† Berkeley, Calif.

of the concrete, as shown by daily cylinders, increased more than 100 per cent. This might be called an "inverted slump test," but it was certainly effective.

L. A. PERRY,* ASSOC. M. AM. SOC. C. E. (by letter).†—This water-ratio specification for concrete has been used by the writer in the construction of approximately 95 000 sq. yd. of concrete pavement during 1926. This method of control was first considered when the writer discussed the subject with Mr. McMillan, in February, 1926. At that time the writer raised many objections to it. However, a fair trial has revealed the pronounced advantages of this control and has established the paper as an extremely important presentation.

Those who realize the importance of the limitation of mixing water and who know that the tendency toward an excessive quantity of water is the prevailing sin in pavement construction, must readily appreciate a specification which induces the contractor through personal financial interest to avoid such excess. Water-ratio control does this in a very practical way and also gives reasonable assurance of a predetermined strength of concrete. Under this specification it also behooves the contractor to select materials of good grading and to avoid stock-pile segregation. The necessity or desirability of inundation of sand ceases to exist. The slump-tube and flow-table are unnecessary. There is no occasion to fix any given sieve analysis or fineness modulus or to specify any given quantities of aggregates. All that is necessary is to specify the water and cement contents of the mixture (or rather the ratio of water to cement) and that the aggregates be clean and of sound grain.

The authors' Table 3‡ gives a wide range of mixtures. The 5 gal. per sack mix has been used by the writer, and test cylinders from this mix checked closely with the strength predetermined by the chart in Table 3. An average of 3 625 lb. per sq. in. was developed by cylinders tested with lumnite caps; and 3 452 lb. per sq. in. was developed by cylinders tested with plaster of Paris caps. Paper moulds were used, cylinders of which do not ordinarily break as high as those cast in machined steel moulds.

The specifications for the work are quoted in part as follows:

"Portland cement concrete pavement shall be composed of a mixture proportioned by parts, as follows: 5 (five) U. S. gallons water, 1 (one) sack cement, and as much fine and coarse aggregate as the Contractor may desire to use; providing, however, that the mixture is at all times, in the opinion of the Engineer, of desirable workability.

"The materials shall be as specified in the 'Standard Plans and Specifications of the City of Longview, Revision of April 1st, 1925'; excepting that the greatest dimension of aggregates shall not exceed ½ (one-half) the depth of the pavement, and that the grading of aggregates within the above limitation shall be optional with the Contractor.

"The amount of mixing water herein specified (5 U. S. gallons per sack of cement) shall include the moisture actually contained in the aggregates

* Constr. Engr., Longview Co., Longview, Wash.

† Received by the Secretary, October 28, 1926.

‡ *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1417.

used; and the amounts of water charged into the batch shall be adjusted in such manner that the moisture in the aggregates and the water charged into the batch shall not, together, be more than the specified water content of mixture (5 U. S. gallons per sack of cement).

"Such adjustments of the water charged into the batch shall be made in accordance with tests conducted by the Engineer, or by his assistants, as frequently as he may deem necessary to determine the moisture content of the aggregates used.

"In the making of such tests and adjustments, the following assumptions shall be made:

"Aggregates weigh 110 lb. (one hundred and ten pounds) per cubic foot.

"The amount of fine aggregate used per sack of cement will be two (2) cubic feet.

"The amount of coarse aggregate used per sack of cement will be four (4) cubic feet.

"It is expressly specified that even though the Contractor may elect to vary the amounts of fine and/or coarse aggregate per batch, the above stipulated amounts of aggregates shall form the basis for adjustment of mixing water charged into the batch (or batches)."

The method of moisture determination was designed for simplicity and definiteness to preclude argument with contractors. All calculations were avoided; a 500-gramme sample of sand was selected by weight and dried, then weighed again on a metric scale. The loss of moisture was read directly on the scale. A double electric hot-plate provided at the bunker and a number of pans enabled the inspector to make tests almost continuously although changes were remarkably infrequent. The time required to make each test was about 5 min. The moisture content of coarse aggregates was determined in the same manner except that it was found desirable to select a sample of somewhat finer grading than was representative in order to offset the tendency of large pieces to dry and lose moisture before the first weighing.

A chart (Fig. 4) was used to determine the quantities of mixing water to be added for various moisture contents of aggregates. For simplicity this chart is based on the total moisture so that it becomes necessary to combine the fine and coarse aggregate moisture, to arrive at the total, when, as in the case of the foregoing specifications, 1 : 2 : 4 proportions are assumed for a basis of adjustments. The total moisture is then the sand percentage plus twice the gravel percentage; for instance, if 4% water is found in the sand and 1% in the gravel, the total percentage used would be 6. Following the chart it is found that the correct quantity of mixing water to be added in this case for a 5-sack batch of 5 gal. per sack concrete would be 17.1 gal. and that the remainder of the 25 gal. (7.9 gal.) is retained in the aggregates. Under some conditions it was believed desirable to sprinkle the gravel stock-pile to keep the moisture content uniform. On isolated work, where this control cannot be kept at bunkers, drying can be conveniently done over an open wood fire, at the immediate site of mixing and deposit.

In the writer's practice it has not been found necessary to make any reservations regarding sieve analysis or fineness modulus of either fine or coarse aggregates or to set any limiting ratio of fine to coarse, as sug-

gested by the authors. Any marked departure from density gradation was keenly felt by the contractor and was remedied very quickly. The bulking effect of sands renders any fine-to-coarse ratio limitation unreliable. It is a comforting fact that the water-ratio theory automatically takes care of sand bulking, and gradation.

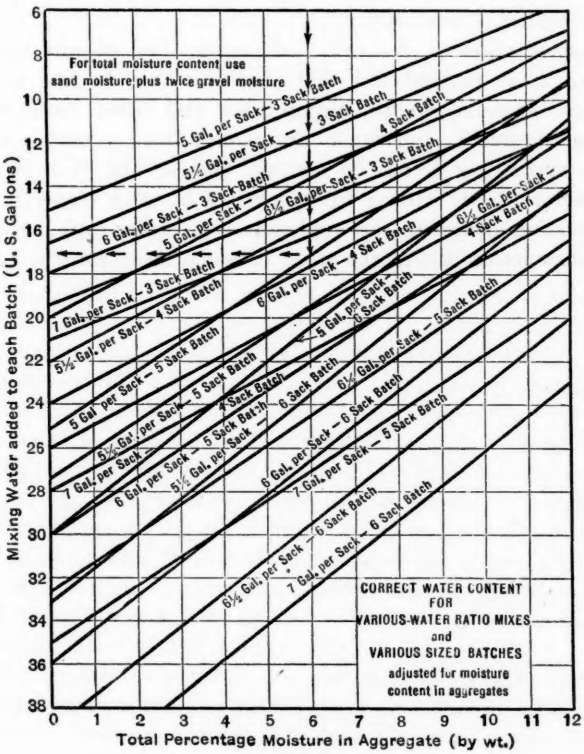

Fig. 4.

It is thought that in the authors' specifications* a typographical error exists where it is stated that "moisture in the aggregate shall be measured by a method * * * which will give results to within 2 lb. for each 100 lb. of aggregate", as this would permit a variation of about 1½ gal. per sack of cement in a 5 gal. per sack mix. This degree of accuracy of moisture measurement is not satisfactory, at least for the best grades of concrete. By the method described in this discussion, moisture measurements were made to within 0.25% by weight (¼ lb. to each 100 lb. aggregate), which permits a variation of ⅛ gal. of water per sack of cement.

The writer has heard many objections to water-cement ratio control, chief among which has been the alleged possible abuse of a seemingly lenient specification, by the use of an excessively lean, dry mix that does not get proper placement. It need only be remembered that economical

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1420.

management of construction does not permit such a thing. In order to save fifty cents in cement by such abuses it would be necessary to spend an extra dollar in labor cost. The writer has seen this demonstrated. It is perhaps a measure of safety, however, to include the reservation in the first paragraph of the specifications quoted in this discussion, thus, "providing however that the mixture is, at all times, in the opinion of the Engineer, of desirable workability."

A full appreciation of the value of this method of mix control will be realized when it is remembered that it eliminates the use of inundation, sieve analyses, fineness moduli, slump-tubes and other consistency gauges, aggregate measurement, and much argument; it is a more simple reliable control than specifying actual or nominal mixes; and it is more equitable to the contractor who is willing to do some of his own inspecting.

This discussion concerns the application of water-ratio specifications to pavement construction where the need of some such control has long been felt.

UNIT STRESSES IN STRUCTURAL MATERIALS

A SYMPOSIUM

Discussion*

By Messrs. J. A. L. Waddell, R. A. Caughey, and E. G. Walker

J. A. L. Waddell,† M. Am. Soc. C. E. (by letter).‡—Mr. Steinman's paper is so comprehensive, so thorough, and so sound that little indeed can be said in discussing it except to endorse the author's views. It may be serviceable, however, to emphasize certain of its salient features.

For the life of him, the writer cannot understand why specifications for structural metal do not invariably demand a minimum "commercial elastic limit" of 35 000 lb. per sq. in. for carbon steel. It is very seldom that the tensile tests of that metal show less than this; and its establishment as a standard minimum would involve no real hardship for the steel manufacturers. Years ago, the writer's bridge specifications§ called for an elastic limit of 35 000 lb. per sq. in. for medium steel; and the metal manufacturers of America in those days furnished many thousands of tons of it with almost no protest and without making any additional charge as compared with the price of the ordinary bridge steel of commerce. There is no valid reason why they should object to doing this to-day.

As to the greatest allowable working intensities for equivalent static stresses on such steel, there may be a legitimate difference of opinion, because the true elastic limit is considerably less than the commercial one—how much less, it is difficult to say. Possibly the difference might amount to as much as 5 000 lb., thus reducing the actual limit to 30 000 lb. per sq. in.

In the old days bridge engineers were of the opinion that the intensity of tensile stress for equivalent static loads should not exceed one-half the elastic limit; and, in conformity with this opinion, the writer then specified 18 000 lb. This figure, like charity, "covered a multitude of sins," such as small eccentricities, secondary stresses (then more or less of a myth or bugbear), and lack of uniformity of metal; but, in the writer's practice, it was not intended to provide for "possible variations in load distribution," "future increase in loading," "inaccuracies of stress analysis," "structural deterioration from neglect," or that German fear-thought "fatigue of metal." Long ago the writer concluded that there is never any fatigue of metal in steel bridges,

* Discussion on the Symposium on Unit Stresses in Structural Materials continued from December, 1926, *Proceedings*.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, October 2, 1926.

§ "De Pontibus," Chapter XVIII.

unless the actual loading, including the impact effect, be fully twice as great as the total load used in the design.

The greater accuracy of modern stress determination, the more scientific detailing of structures that has slowly been evolved, and the exclusion of most of the objectionable features of former bridge designing, make it perfectly proper to-day to fix the intensities of working stresses for bridges at a higher percentage of the elastic limit than formerly. Instead of the 50% then used, it is now perfectly safe to assume 60%, or even a little more. The better the profession is informed about the true science of bridge designing, the greater may this percentage be taken with safety. A few years hence it may be proper to adopt 65%; but, in the writer's opinion, that should be about the ultimate limit. Taking 30 000 lb. as the true (not the commercial) elastic limit, the working intensity to-day would be 18 000 lb. and that for the future, 19 500 lb., which might, perhaps, eventually be raised to 20 000 lb.

It is, of course, theoretically proper to stress steel to the same limits in bridges and in buildings, as the working intensities in both are based on equivalent static loads; but actually it is a little safer to increase stresses in buildings than in bridges, for the reasons that in the former the assumed live loads are less likely to be reached, and the impact effects are generally small and infrequent. The writer, therefore, would not be opposed to using to-day a stress of 20 000 lb. in tension for buildings, but would seriously object to more than 18 000 lb. for bridges.

In respect to the question of column formulas, the writer used to make a distinction between compression chords and all other struts. For fixed ends he adopted $18\,000 - 70\,\frac{l}{r}$ for chords, and $18\,000 - 60\,\frac{l}{r}$ for other struts. An average of these is $18\,000 - 65\,\frac{l}{r}$.

The latest proposed formula, namely, $16\,000 - 8\,E$, is equivalent to $16\,000 - 80\left(\frac{l}{r} - 50\right)$, or $20\,000 - 80\,\frac{l}{r}$. For $\frac{l}{r} = 100$, the average of the writer's old formulas gives an intensity of 11 500 lb., while the latest formula gives 12 000 lb.; hence, there is no material difference between the old and the new methods, because 100 is a fair average for the slenderness ratio.

Mr. Steinman has condensed to the limit the principles that in steel building specifications should govern the writing of the various clauses for the determination of the sectional areas of metal when he states that:

"Any anticipated increase of load should be covered by increasing the assumed live load, not by reducing the basic unit stress. Any impact or shock effect should be covered by adding a proper dynamic increment to the live load stress, not by diminishing the basic unit stress. Structural deterioration should be minimized by proper detailing and maintenance, and, where inevitable, should be covered by a suitable increase in the thickness of metal in the affected parts, not by a general reduction in the basic unit stress. Only the uncertainties in the strength and behavior of the material, and such other elements as cannot be allocated elsewhere, should be included in the margin that determines the basic unit stress."

These principles agree exactly with those which have governed the writer's practice during the last thirty years.

The weaknesses in most steel buildings lie mainly in the detailing. This is not often done with the thoroughness and the science that characterize the detailing of modern steel bridges, but certainly it should be so done. The torsion from tornadoes and the racking from earthquake shocks should be guarded against effectively in all first-class building construction.

The Engineering Profession needs an exhaustive treatise on the design of steel buildings, which would enable any competent engineer to evolve structures that would be safe under practically all conditions. It should contain specifications for both design and construction as thorough and complete as the best bridge specifications yet written; and every kind of detail that is likely to be needed in any steel building should be elaborately illustrated by drawings and explained by ample dissertation. Such a book would be invaluable to structural engineers both in the United States and abroad; hence it is to be hoped that some experienced and competent structural specialist will accept this suggestion and prepare this much needed treatise.

R. A. Caughey,* M. Am. Soc. C. E. (by letter).†—It would seem from the papers, on "Unit Stresses in Structural Materials", as well as from similar papers and opinions, that working stresses in steel and concrete should be increased; and in view of the fact that methods of manufacture are much improved, this change would seem very logical. However, if this revision is made in any code or specification there are certain questions that should receive attention in order to safeguard structures built under such a code or specification.

First.—Specifications should be prepared more carefully. This is especially true for reinforced concrete construction for which, in many instances, the specifications are very inadequate as regards materials entering into the concrete, methods of combining these materials to make good concrete, and provisions for tests of the final product. These difficulties may be overcome by specifying proper qualifications and tests of individual materials, and by stating that they shall be combined in a scientific way to make concrete in which certain strengths shall be assured. The day for arbitrarily specifying a 1 : 2 : 4 mix and paying no attention to the quantity of water to be used, the consistency, the gradation of the aggregates, etc., has passed. These objections, or similar ones, can hardly be urged against steel, as steel manufacture is in the hands of experts and the tests are more adequately set forth in standard specifications.

Second.—It would seem that if working stresses are to be increased, better "field control" should be insisted upon. This would apply to both steel and concrete and for steel, better fabrication than is sometimes done should be insisted upon. It is hardly worth while to state that many structural members, on account of careless fabrication, are unable to carry, without a heavy dependence on the factor of safety, the loads applied to them. Getting around

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† Received by the Secretary, November 1, 1926.

these difficulties would call for "field control" in the form of careful shop and erection inspection.

Concrete perhaps suffers more from lack of inspection than steel, because of the temptation to increase profits by saving cement; by wide variations in materials that are assumed by the specification writer to have no variation; and by ignorance on the part of the contractor who thinks that concrete should be mixed with water (and "lots of it"), but not with brains. The overcoming of these difficulties will call for constant inspection, not a daily visit by the architect or engineer; a constant checking of materials by tests; a changing of mix to meet changes in aggregates; and testing of the final product. These requirements may seem unreasonable and expensive, but it has been noted that attempts have been made in recent years on many large contracts to meet them by using the "water-cement ratio" method of field control, among which may be mentioned the Portland Cement Association Building of Chicago, Ill.,* and the work of the Bronx Parkway Commission of New York City.†

Third.—An increase of working stresses should call for greater care in stress computations, notably secondary stresses. Structures have been designed in which the secondary stresses have exceeded the primary ones at certain points. A case of this kind should not be passed by without notice; and, in order that secondary and other stresses of an indeterminate nature may be more accurately calculated, field work on stress measurement as well as development of higher theory should be encouraged.

Dean Turneaure calls attention‡ to the fact that lower values of unit stress have been used in bridge design to allow for future increases in loading. This is one way to care for such a situation; but every one will agree that if higher unit stresses are allowed, this same allowance could be made in a different and more scientific manner, namely, by an increase of loads used to compute stresses.

Mr. Steinman§ in a different manner, sounds the same note as Dean Turneaure, in his statement that anticipated increases in building loads, effects of shock, impact, etc., should be covered by increasing the live load rather than diminishing the basic unit stress. He also calls attention to the common practice of neglecting the effects of such items as dead weight of columns, column coverings, etc., which, of course, should be considered when higher unit stresses are used.

It may appear, from the foregoing, that the writer does not favor increases in unit stresses, but such is not entirely true. However, if unit stresses are to be increased, the capacity of the materials to be stressed and the manner in which they are combined to form structural members must receive more attention. In addition, the actual stress that is carried by certain members must be more carefully determined. In other words, scientific methods must be used by all who have anything to do with construction, beginning with the

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1418.

† *Engineering and Contracting*, April, 1925, p. 866.

‡ *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1424.

§ *Loc. cit.*, p. 1432.

specification writer and the manufacturer of the materials, and ending with the final acceptance of the structure.

There are many reasons to believe that these hopes can be realized on large projects designed by engineers who can, and will, do the work scientifically and in a painstaking manner, and constructed by contractors who have an appreciation for engineering science; but there are many places where these accomplishments cannot be expected as yet. For this reason, any upward revision of unit stresses should be undertaken only where the construction and design standard will warrant it.

The writer does not feel that steel stress should be raised to 20 000 lb. per sq. in. with yield points as at present and without a guaranty of better fabrication than is furnished on some jobs. More care should also be guaranteed with respect to distribution of steel in cross-sections of members. It would seem that, at present, 18 000 lb. per sq. in. is a more reasonable value and that it should be adopted only where requirements set forth in the beginning of this discussion can be met. It would also seem that 800 lb. per sq. in. is high for 2 000 lb. concrete unless the 2 000 lb. ultimate strength can be realized with almost 100% surety. It is interesting to note, in this connection, that W. F. Welsch, Division Engineer, Bronx Parkway Commission, reports* that of thirty-two samples of concrete taken from field operations on three different jobs, only two tested below strength values for which they were designed by the "water-cement ratio" method. This would seem to indicate that ultimate strengths can be forecasted in concrete almost as closely as in steel, if all contractors take up scientific proportioning as cheerfully as does John G. Ahlers, M. Am. Soc. C. E., who makes the following prophecy:†

"Within a few years design of concrete will be considered a simple matter. It will be this: First, the engineer will use a fixed water-cement ratio for any given strength—and get it; second, the contractor will, after fixing his water-cement ratio, be let alone and allowed to add all the sound, best-graded material he possibly can and still work his mix. Let us hope that all professional and practical people will help us bring that happy day to pass as soon as possible".

The 12% shearing stress seems excessively high; in other words, it does not seem reasonable to assume that horizontal bars with special anchorage can be depended on to increase shear resistance of beams 100 per cent.

E. G. WALKER,‡ M. AM. SOC. C. E. (by letter).§—The writer strongly supports the point of view put forward in Mr. Steinman's paper.‖ Safety and efficiency in design can only be achieved by analyzing the stresses developed in a structure to as close an approximation as knowledge of the subject permits, and then allowing the maximum working stress considered to be safe for the particular type of loading involved.

* *Engineering News-Record,* Vol. 95, 1925, p. 630.

† *Loc. cit.,* Vol. 95, October 22, 1925.

‡ Care, Maxted and Knott, Ltd., London, England.

§ Received by the Secretary, November 27, 1926.

‖ "Unit Stresses in Structural Steel (for Buildings)," *Proceedings,* Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1432.

The argument frequently adopted, that a low working stress is taken in order to allow for uncertainties in loading, is, in the writer's opinion, weak. It is, in many cases, a cover for inadequate knowledge and disinclination to analyze designs fully, rather than the expression of a definite principle. It is unfortunately a fact that in the design of the structural framing of buildings there is a greater proportion of empirical assumption than in that of bridges. Buildings are normally designed for certain specified loads per unit area; these vary in accordance with the use to which the particular area of the building is to be put. Horizontal loading from wind is calculated at so much per unit area on certain specified surfaces with perhaps a variation of the unit pressure between the lower and the higher part of the building. As frequently as not, at the time the building is being designed, its future internal arrangements are only known in a general way. Heavy partitions and other fittings are ultimately placed in positions which are decided after the building is erected.

The only way to provide for this condition is to design the structure for such uniform load as would be adequate to cover the highest stresses developed by these unknown concentrations. It follows, however, that the actual stresses developed in floor members vary more or less considerably from the calculated stresses according to the degree in which the actual use of the building approximates to the imposition of a uniform live load. It is generally agreed that the uniform loading basis is an adequate one in that it provides for computed internal forces and moments which are at least as high as those actually developed in structures after completion, and in many cases are considerably higher.

Wind forces are treated in a highly empirical manner in civil engineering design and the calculated internal forces and moments in the various members of the structure of the building due to wind pressure must of necessity bear only a shadowy relation to the actual forces and moments developed during a wind. Investigations have shown that the distribution of air pressure over the surface of a body exposed to a uniform air current is very irregular and varies considerably with the shape of the object and its orientation to the current. In most cases the area of positive pressure is a relatively small proportion of the exposed area of the object, and the greater part of the resistance of the body is due to the existence of large areas over which negative pressures are developed. Other investigations have shown that the distribution of air pressure and velocity in an ordinary wind is extremely irregular and uniformity cannot be relied on even between two points situated only a few feet apart. This adds considerably to the complicated nature of the distribution of pressures and suctions over the exposed surface of an actual building in a wind. In the present state of knowledge it is difficult to estimate with any degree of exactness what this distribution is, and, therefore, recourse is had to the early assumption of a uniformly distributed pressure over the surfaces out of the direct line. How approximate the forces and moments calculated on this assumption are to those actually obtaining is a matter which, the writer believes, is under investigation at present.

The writer has enlarged on these two points in relation to the loading of buildings in order to emphasize the argument that the computed forces acting on members of the framework of a building are at best only approximate estimates of the maximum for which the designer of the structure considers it desirable to provide. The more closely a scientific study of conditions enables a forecast of the actual loads in the members, the greater will be the liberty given to the engineer to settle the stresses on a scientific basis. It is generally agreed that where the load on a member is accurately known, proportioning it on the basis of a maximum stress of 16 000 lb. per sq. in. is unduly conservative and, therefore, wasteful of material. In the discussion of this subject, instances have been cited of computed unit stresses from 18 000 to 26 000 lb. per sq. in. in structures that are thoroughly safe. Researches on the subject also show that high unit stresses can be used with safety, so far as the strength of the material is concerned. It is, however, impossible to allow indiscriminate use of high unit stresses unless it is certain at the same time that the forces and moments which the members of a structure are designed to resist are actually those, both in range and maximum, that the members will be called on to take when the structure is in use.

The writer's plea is, therefore, for a more exact scientific determination of the magnitude and nature of the internal forces developed in structures as the most important step required at present for the advancement of structural design. Of recent years there have been extensive investigations that have shed considerable light on the behavior of structural materials under all kinds of conditions of loading and, therefore, engineers are in a much better position at present to assess the maximum stresses which a particular material can withstand indefinitely under a particular set of conditions. Without adequate knowledge of the actual conditions of the case, however, it is unsafe to stress the material to these limits.

WATER-PROOF MASONRY DAMS

Discussion*

By Messrs. H. de B. Parsons, Alfred D. Flinn, J. B. W. Gardiner, T. Kennard Thomson, V. Bernard Siems, B. F. Jakobsen, and Eugene E. Halmos.

H. de B. Parsons,† M. Am. Soc. C. E.—All dams are intended to be water-tight, that is, tight against leakage, but not necessarily tight against percolation of water. Earthen dams and rock-filled dams are not expected to be tight; but as they are not "masonry" dams, they are without the scope of this discussion.

In broad conception, there are three elements entering into a design for a dam, that is, a water-tight structure, a mass to give strength, and a shape to effect economy in construction. Masonry dams are built of stone, or concrete, or both in combination. They are intended to be water-tight, and an effort always is made to make them water-proof. This effort is carried as far as considerations of economy will permit. Large dams of gravity cross-section, as illustrated in the paper, almost always are made of stone or of stone facings with cyclopean concrete backing. The Spier Falls Dam across the Hudson River, for which the speaker made the designs, was of the latter character. Artificial stone is used at times in place of natural cut stone.

To build an effective water-proof membrane in a stone dam would be extremely difficult. The stones overlap and break joints, the shapes of the sides and backs of the stones are irregular, and they are placed so as to interlock one with another. To construct in the masonry a water-proof membrane, such as is mentioned in the paper, would seem to be a difficult operation.

A water-proofing membrane of lead, copper, zinc, or cloth, as described by the author, therefore, seems to be limited to mass concrete structures and not to those of stone or concrete blocks. The title of the paper would appear to be misleading, if the speaker's conception of the suggested membrane be correct. The section of an all concrete dam is usually different from the gravity sections shown in the paper. Arch dams, multiple-arch dams, buttress dams with aprons, and similar designs achieve savings in concrete that are as great as or greater than the percentages mentioned by the author. Economy in the quantity of concrete used, and in the cost of its placement, depends on the shape and form of the section selected. The cost of wooden forms must not be overlooked, and the additional cost of

* Discussion on the paper by W. Watters Pagon, M. Am. Soc. C. E., continued from December, 1926, *Proceedings*.

† Cons. Engr., New York, N. Y.

forms for the insertion of a water-proofing membrane, as illustrated, might be a serious factor.

Concrete will not hold fast to and bond with smooth flat metallic surfaces; and, in the speaker's opinion, it is wise that the author has suggested the use of diagonal tie-rods. These rods, passing through the water-proofing membrane, will tie together the concrete masses separated by the membrane.

The suggested membrane materials have different linear rates of expansion. Taking concrete expansion as 100, copper has a relative expansion of about 121, lead about 188, and zinc about 203. In other words, the expansion under temperature changes of a copper membrane would be more than 20% greater than concrete, and that of a lead or zinc membrane even more. The difference in expansion of concrete and a metal membrane will result in rupture or separation of the concrete mass from the metal surface. The ranges of temperature to which the water-proofing membrane might be exposed in a climate like that around New York, is roughly 30° Fahr., say, 32° for the minimum temperature of the water, and 62° for the maximum. For that part of the membrane above the water-line, the range of temperature would be greater.

Since the 3-ft. concrete section back of the water-proofing membrane might separate from and act independently of the main concrete mass, it is questionable whether engineers would include this section in their calculations for strength of such high dams as those illustrated in the paper.

Practically all stone dams (whether natural or artificial) are more or less porous. Luckily the tendency is for silt to find its way into the pores of the stone, and gradually close them. In a natural stone or artificial stone dam, the leakage, if any, is through the joints. After a dam is built, joints between stones and, in a concrete dam, construction and expansion joints need careful watching. The same care would be necessary to watch the membrane, which must have many joints between the copper, zinc, or lead sheets. These metal joints would exceed in number the joints in a mass concrete dam, where they are limited to expansion and construction joints.

Although ordinary concrete is porous it can be made almost water-tight by proper proportioning or working of the ingredients. Therefore the thought arises, would it not be cheaper, that is, more economical, to regulate closely the aggregate, cement, lime, and water of the mix, than to construct a water-proofing membrane as proposed?

There are applications which can be applied to the surfaces of concrete that will almost render it water-tight or water-proof. In the speaker's experience the greatest benefit from these applications results from the careful inspection of the surface of the concrete, while smoothing it, brushing it, and filling up the holes left by the forms.

In a concrete dam it is difficult to keep water out of construction joints and shrinkage cracks. When water reaches the reinforcement, the metal rusts and, in rusting, produces an expansive force that breaks the concrete. This trouble is usually noticed between "wind and water", that is, where the reinforcement is alternately wet and dry.

Ice is another serious cause of trouble. The impact of ice floes breaks the hard surface of the concrete, and opens channels for water to enter into the mass. If it does, the water then will reach the reinforcement.

The author assumes the foundation to be tight, and he states that "if not, why build on such a foundation?" Unfortunately, the engineer often has to build a structure on a pervious foundation. If the foundation is pervious, or if water enters under a dam through fissures in the rock, either under or at the sides, there will be uplift at the base which a water-proofing membrane will not prevent. With an uplift pressure it would seem wiser to expend money in trying to control the uplift, rather than to construct a water-proofing membrane, such as described, which would be difficult to make tight at the joint with the foundation material.

What the speaker has said should be taken not so much in criticism of the paper as a statement of the difficulties of construction and maintenance as they appear from the standpoint of design and construction in the field. It is hoped that mentioning facts which may seem in opposition will encourage debate and discussion. To design and construct a really water-proof dam would undoubtedly be a great achievement.

ALFRED D. FLINN,* M. AM. SOC. C. E.—Such experience as the speaker has had in designing and building dams leads him to doubt seriously the practicability of successfully incorporating in a dam a water-tight diaphragm or membrane, as proposed by the author. The placing and safeguarding of the membrane and anchors (Fig. 1†) are not compatible with economical methods of building massive bodies of concrete, cyclopean, or other kinds of masonry used for large dams.

Other considerations than prevention of uplift led to the provision of drainage pipes, wells, and galleries in some well-known dams. One purpose was to reduce or prevent the slow local deterioration of the masonry by the leaching action of water wherever it could seep through. Another purpose was to prevent or reduce disfiguration of the down-stream face of the dam by seepage. Still another purpose was to prevent suspicions as to security sometimes aroused in lay minds by the appearance of seepage on the face of the dam. Some reduction of uplift may be gained also, but drainage passages cannot be depended on permanently because the leaching and depositing of salts from the masonry tend to seal the designed and accidental passages.

J. B. W. GARDINER,‡ ASSOC. M. AM. SOC. C. E.—The speaker would like to discuss one point that has already been mentioned several times, that is, the method of fastening the protecting face against both the water-proofing and the main body of the dam. The author's design (Fig. 1†) shows the water-proofing to have been punctured in many places by the rods used in tying the two parts of the dam together. There is a great element of danger, particularly where heavy hydrostatic heads have to be combatted, in puncturing a water-proofing blanket and, as already pointed out, it has not proved

* Secy., United Eng. Society and Director, Eng. Foundation, New York, N. Y.

† *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1574.

‡ Pres., J. B. W. Gardiner, Inc., New York, N. Y.

satisfactory in practice. It is the speaker's opinion that any water-proofing plan should be so designed as to avoid such puncturing.

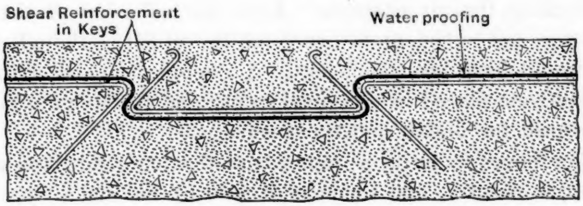

FIG. 4.

In this case a design that suggests itself as accomplishing the desired end is shown in Fig. 4. Keys, as shown, would be cast in the face of the main body of the dam, running vertically to facilitate the placing of the concrete face. The strength of shoulder necessary to take care both of anchor ice and of the tendency to overturn, could be calculated and shear reinforcement placed as shown. The water-proofing would be carried down into the key and the entire mat over the face of the dam could be made a continuous unbroken sheet, without breaks or punctures. The speaker does not believe that the cost would exceed that of the author's design, while the efficiency would be vastly increased.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—It is hard to conceive of any membrane water-proofing under a dam that might not do more harm than good.

One reason for this is that most membrane water-proof materials are liable to be destroyed in time—leaving a space for the water to spread—thereby greatly increasing the uplift. Copper itself will deteriorate under certain conditions, and even if it did not, it could not be hammered down on the bed-rock so that spaces would not be left beneath, which would be filled with water.

In New York, bed-rock is practically impervious, but fissures are frequent with a full head of water, which, if allowed to spread under the base, would give an uplift proportional to that spread.

There was a case in the West, where, if the entire dam had been water-proofed from the outside—top, bottom, and sides—it might have been saved, regardless of uplift. That was a steel dam, the first cost of which was 10% less than a masonry gravity dam. A few months after construction, the dam collapsed, and it was then found that a 10-lb. hammer had been almost completely destroyed by the acids in the water.

Theoretically, the worst conditions can be calculated, provided a few important factors are not overlooked; but to try to guess at the actual uplift and ice pressure requires more than a calculating machine. For example, all the Hudson River Tunnels are lighter than the water displaced, and it cannot be proved that they will stay "put", nor the reverse.

* Cons. Engr., New York, N. Y.

Again, a pneumatic caisson was founded on rock and had its air chamber filled with concrete when it broke loose, rose, and moved, due probably to the fact that the concrete had been stopped 4 ft. below the intended elevation before sealing the air chamber. Even the adhesion of the concrete to the bed-rock was not sufficient to overcome the uplift, although another foot of concrete would have been sufficient. A pneumatic caisson in New York had 4 ft. of concrete placed in the air chamber and the weights taken off with an utter disregard for uplift, and up it came.

The speaker hopes to build a dam where the ice conditions are probably the worst in the country, and to make it safe by having a slope on the up-stream side so flat that the ice cannot create any pressure against the dam, and with a similar flat slope on the down-stream side so that the ice will slide down into deep water, instead of, as usually happens, dropping from the top of the dam and undermining the foundation.

V. Bernard Siems,* Assoc. M. Am. Soc. C. E. (by letter).†—In this paper, Mr. Pagon has proposed an interesting subject. His treatment is careful, complete, and novel. However, certain questions immediately arise as to the desirability of the plan which he has suggested. Whether to sacrifice mass in dam structures for the purpose of reducing cost by substituting an absolutely impervious up-stream facing consisting of a membranous coating is, to the writer, an open question.

The author's design inserts in the dam structure an artificial cleavage plane, an opportunity for mass faulting, for it is questionable whether or not the face mass could be anchored to the body of the masonry to prevent a vertical plane of excessive weakness in shear. With the separation of the face mass from the masonry body by a frictionless membrane, the two masses, notwithstanding the anchors, could not be considered as acting monolithically. The writer also doubts the permanent tightness of the membrane, especially at the point where the bars pass through it, or at any puncture, for such points would be difficult to seal completely. He also believes the coating would be difficult to apply.

Certainly efforts should be made to obtain impervious concrete with a hard outer surface resistant to frost action. Concrete deteriorates when water penetrating beyond its surface dissolves the calcium compounds, and permits frost and ice action to disintegrate the concrete mass mechanically. The method of building the facing is not entirely clear. If built after the main body of the dam, no doubt the thin facing would be more expensive than its volume would indicate. A well-designed and constructed concrete dam should not have any large percentage of its interior subjected to uplift by water. No doubt a dam the face of which has been constructed of a rich mix, with its inner mass of an 1:3:6 mix, would absorb in the interior mass any small leakages that pass through the exterior base without exerting uplift. If, as Mr. Pagon suggests, inspection and drainage galleries are built, the upward pressure will be relieved.

* Water Engr., City of Baltimore, Baltimore, Md.

† Received by the Secretary, November 11, 1926.

The chief trouble from uplift is in or under the foundation. The greatest damage from uplift would occur in case there were subterranean fissures beneath the dam into which water might find its way. Large dams should always be provided with porous drain-tile discharging into the gallery for the purpose of carrying off any leakage that might pass the outer and more impervious concrete.

Where large dams are built in thickly populated territory, an ample factor of safety should always be provided. The writer prefers to rely on mass rather than on a membranous water-proofing near the face of the dam, especially as the danger point may be beneath the dam, as is true in some parts of the world—India and Egypt, for instance—instead of in the interior mass.

He would rely, therefore, on mass and a good quality of concrete that can be secured by restricting to proper limits the quantity of water used, by the prevention of loss of cement, by the insurance of a proper mix, by proper tamping and keying, and by protection from injury by jar or frost.

B. F. JAKOBSEN,* M. AM SOC. C. E. (by letter).†—The Los Angeles County Flood Control District has under consideration a gravity dam nearly 500 ft. high on the San Gabriel River, for which $25 000 000 was voted in 1924. The present design, known as Design R, was prepared by the Designing Engineer for the Flood Control District, Mr. S. M. Fisher and has received the approval of two consulting engineers, John D. Galloway and D. C. Henny, Members, Am. Soc. C. E. This design provides for 50% uplift at the heel and zero uplift at the toe.

A considerable saving, approaching 15%, or $3 750 000, can be effected, judging from the author's Fig. 3‡ and, therefore, the merit of the proposed remedy seems well worth investigation.

The author states:§ "Of course, it is assumed that the foundation bed will be tight." Unfortunately, however, if uplift is at all likely to be present, the writer's experience is that the joint between the rock and the concrete is the most dangerous point. If the proposed water-tight membrane costs as much as designing for uplift, it would most likely not be proposed; its only advantage is its cheapness.

The author states:‖ "Since, therefore, no important dam would be built nowadays without considering uplift, the whole question of water-proofing appears to be important." The writer wishes to call attention to two major dams, the Don Pedro Dam and the Exchequer Dam, both located in the San Joaquin Valley, California, which have drainage galleries and cut-off walls with drains behind them, but otherwise have no provision for uplift included in the design. It was evidently assumed that the provisions made were so successful as to make it safe to assume that there could be no uplift. A. J. Wiley, M. Am. Soc. C. E., was the Consulting Engineer for both projects. Whether uplift could ever be present may be a

* Engr. in Chg. of Dams, Los Angeles County Flood Control Dist., San Fernando, Calif.

† Received by the Secretary, November 18, 1926.

‡ *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1576.

§ *Loc. cit.*, p. 1575.

‖ *Loc. cit.*, p. 1574.

mooted question on which competent engineers may disagree, but to assume no uplift is always less safe than to design for some uplift.

It may be well to bear in mind that there are various degrees of probability, as, for example, it is highly probable that 1 cu. yd. of concrete will always, weigh, say, 2 tons; it is less certain that it will also have the estimated strength, and still less certain that tension can be transmitted across contraction joints.

There is some danger that the membrane may be damaged during construction and that water may accumulate at the up-stream face of the membrane and possibly detach the concrete from it. In computing the stresses in a gravity dam provided with such a skin, the writer would insist that the section be reckoned from the skin down stream and that the 3 ft. of concrete lying up stream from the membrane be excluded.

When considering what can be done to lessen or eliminate uplift, it should always be had in mind that to design a dam of a sufficiently heavy section is the best and most positive remedy. To provide drains may be considered in many instances at least as a partial remedy, but there is always the danger that drains may clog, while there is not much danger that the material will lose in weight. As an illustration, in a paper by F. E. Dolson and W. L. Huber, Members, Am. Soc. C. E., entitled "Multiple-Arch Dam at Gem Lake on Rush Creek, California",* no account appears to have been taken of the fact that the drain pipes are likely to become closed by freezing. Fig. 8† shows a porous precast block with 4-in. tile pipe connected, but a specific question by the writer in his discussion‡ as to what provisions had been made to prevent the closing of drain pipes by freezing and what the factor of safety would be in case this should occur, remained unanswered, except for some evasive general remarks.§ The writer, therefore, feels entitled to conclude that nothing was done to prevent such clogging; and yet these drain pipes are the essential and possibly the only remedy applied to prevent a dangerous condition due to uplift. To assume outright that uplift cannot occur, or to assume that drain pipes will always work, may sometimes amount to the same thing, and the engineer should guard against just such a contingency.

The author obtains his saving by comparing a design good for a two-thirds full hydro-static pressure at the heel, but without a skin, with one provided with a water-tight skin, but otherwise having no allowance for uplift. The writer cannot agree that these two designs are even approximately equally meritorious and, moreover, he would always design a high gravity dam so that there could be no reasonable question about whether or not tension exists in the up-stream face when the reservoir is as completely filled as it is ever likely to be.

The middle-third theory assumes that horizontal sections remain plane during deformation, which is certainly not an exact assumption, but on the other hand is not likely to be far wrong. The general bending theory

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 713.

† *Loc. cit.*, p. 728.

‡ *Loc. cit.*, p. 763.

§ *Loc. cit.*, p. 788.

assumes that normal planes remain plane and this assumption excludes the one that horizontal sections remain plane, since these are not normal but oblique to the axis of the structure.* Moreover, the assumption that normal sections remain plane, has been verified, as far as the writer knows, only for small sections, say, those of a few inches in thickness and is true only when no shear, or a uniform shear, exists. The assumption may not be exact and some caution is justified when applying it to a section several hundred feet thick and with unevenly distributed shear.

As far as the writer is aware all stress determinations made to date assume either that horizontal sections remain plane,† or that the normal planes remain plane.‡ They all disregard the warping of the section due to shear stress and the influence that this may have on the distribution of the bending stresses.§ For high dams designed without uplift, this influence may not be negligible. Also, experiments made to verify the fundamental assumption that normal planes remain plane, have been made on beams of uniform section; but a gravity dam is not such a beam and moreover it is fixed along an oblique plane, which is itself deformed by the stresses imposed on it.‖ Also, swelling due to water soaking, shrinkage, temperature variation, irregularity of foundation, etc., will have some influence on the stress distribution.

These various defects in the ordinary bending theory are pointed out for the reason that the assumption of considerable uplift enables the designer to neglect these errors and to treat the ordinary bending formulas as approximate, which they are. The stress in the heel of a dam when the reservoir is full is due to the difference between the stress when the reservoir is empty and the bending moment produced by the water when the reservoir is full. For a dam 500 ft. high, with a vertical up-stream face and a triangular section, designed with no uplift, and with concrete weighing 144 lb. per cu. ft., the stress in the up-stream face when the reservoir is empty is 500 lb. per sq. in., and the tension produced by the bending moment due to the water load is also 500 lb. per sq. in., so that the resulting stress in the up-stream face with reservoir full is zero. It is evident that if the bending stress is under-estimated by 5%, due to a stress distribution different from that assumed, a tension of 25 lb. per sq. in. exists in the up-stream face of the dam. This would not in the writer's opinion be a safe structure even if uplift could not occur. If uplift had been included in the design, however, the error in the ordinary bending theory would normally only reduce the compression in the up-stream face and would be no very serious matter. This same error would

* "Stresses in Multiple-Arch Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 289 and 292, and also the discussion, by William Cain, M. Am. Soc. C. E., p. 316.

† See "Stresses in Masonry Dams," by Ottley, Brightmore, Wilson, Gore, and Hill, *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session 1907–'08, Part II. Also, "Stresses in Masonry Dams" by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 208.

‡ "Formes et Dimensions des Grands Barrages en Maçonnerie," by M. Résal, *Annales des Ponts et Chaussées*, II, 1919.

§ See discussion by J. P. J. Williams, M. Am. Soc. C. E., on a paper by H. S. Prichard, M. Am. Soc. C. E., entitled "Faults in the Theory of Flexure," *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 932.

‖ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, Fig. 45, p. 1254.

also effect the maximum stress in the down-stream face, but that would be of but slight importance, since it may be assumed that the error is not large.

Until the actual stresses in high gravity dams have been determined from experiments and theory, engineers will do well to bear in mind the possible influence of the inaccuracies in the fundamental assumptions. The proposed water-tight membrane may keep water out of the structure—except near the foundation—but it does not provide any protection against the factor of ignorance arising from the fact that the fundamental assumptions are known to be only approximations.

EUGENE E. HALMOS,* M. AM. SOC. C. E. (by letter).†—The history of the development of the rational design of gravity dams, no matter how brief, should not omit mention of the work done by Maurice Levy and Karl Pearson. No gravity dam of importance, built in the last twenty or twenty-five years, was constructed without due consideration having been given in the design to the principles enunciated by these men. Their names are as intimately connected with the status of the present knowledge on stresses in gravity dams as those of Lord Kelvin, Crookes, and Steinmetz with the development of modern electrical engineering.

Modern dam design, based on the researches of Levy and Pearson and on model experiments demonstrating the truth of their conclusions, recognizes the fact that Rankine's criterion, namely, that the resultant of the forces must fall within the middle third of the section, is not sufficient to prevent the occurrence of tension in the mass of the dam. There may be tension in the masonry on planes other than horizontal even if all the stresses on the horizontal planes are compressive. To prevent tension it is necessary to design the dam so that there will be a substantial compression at the up-stream edge of any horizontal-plane section. This compression must not be less than the intensity of the water pressure at the depth of the section investigated. This criterion, in the case of high dams, results in a materially heavier cross-section than that complying with the middle-third rule only, and such an enlarged section is necessary without regard as to whether or not the back of the dam is treated with water-proofing paint or is supplied with a water-tight membrane. On the other hand, compliance with this criterion will be entirely sufficient to make the dam safe as long as the uplift at the bottom is given proper consideration in the design.

The writer believes that the water-proofing of gravity dams for the purpose advanced by the author is entirely useless and may be a source of danger if the section is diminished from that necessary to prevent tension from occurring in the dam. In no case would he approve of a design which would consider the concrete covering of the proposed water-proof membrane to act as part of the dam.

The writer cannot help but criticize a tendency, fortunately not too frequent among engineers, to try to save money by the use of makeskifts at the risk of endangering the safety of important structures on which the life and prosperity of many human beings depend.

* Chf. Engr., Parklap Contr. Corporation, New York, N. Y.

† Received by the Secretary, November 30, 1926.

THE DESIGN, CONSTRUCTION, AND OPERATION OF A SMALL SEWAGE DISPOSAL PLANT

Discussion*

By Webster L. Benham, M. Am. Soc. C. E.

Webster L. Benham,† M. Am. Soc. C. E. (by letter).‡—No matter how carefully a sewage disposal plant is designed nor how complete the general plan for treating sewage, it will be of little avail if the plant is not properly operated after it is turned over to the municipality. The successful operation of a sewage disposal plant, particularly in small cities and towns, is a bugaboo to the designing engineer. Until almost complete jurisdiction is given to State authorities requiring careful and constant attention by qualified operators, engineers cannot expect sewage disposal plants to operate successfully and to show the results for which they are designed.

The State of Oklahoma has on its statutes a law prohibiting the pollution of its streams, but the "teeth" of the law were removed by the Legislature when the original bill was presented. The result is that the State has no means of forcing a municipality to operate consistently and efficiently its sewage treatment works. It is almost appalling to see the damage done year in and year out to the sewage disposal plants, particularly in the small towns, by careless officials who seem to think that such plants are something to be built and forgotten. The writer knows of several plants in the State where contact beds and sprinkling filters have become beds of weeds and sunflowers, and where the plants have been almost dismantled by mischievous youngsters. There are even cases where the plants themselves have been abandoned and sewage is being emptied directly into ravines and streams rather than have any one "bothered" with any responsibility for the operation of the plant.

Such a condition would not and could not exist if the State Board of Health had proper jurisdiction and was protected by satisfactory laws and regulations. It is hoped that this condition will not continue long. The change for the better, however, will be due to the awakening of the people (and possibly the engineers) to the fact that investments of cities should be given proper protection and the health of the community preserved.

The writer has always made it a point to file with the city a complete schedule of operation of a sewage disposal plant. In the case of Imhoff tanks, these rules show ways and means and the tools necessary to clean the slides

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* Discussion on the paper by Franklin Judson, Jr., Jun. Am. Soc. C. E., continued from December, 1926, *Proceedings*.

† Pres. and Chf. Engr., Benham Eng. Co., Kansas City, Mo.

‡ Received by the Secretary, November 16, 1926.

and slots of the sedimentation chamber so that the scum in the vent chambers can be broken up or removed. General instructions are also given relative to the removal and drying of sludge and for the resurfacing of the sludge drying beds.

The operator of every sewage treatment or disposal works should be provided with conical glass graduates for testing the quantity of suspended matter in the raw and settled sewage, so that from these observations the efficiency of any installation may be roughly determined. Tests for stability of the effluent and other indicative tests of a practical nature should be conducted daily by the operator. Such a thing is almost unknown in Oklahoma, simply due to the fact that few municipalities realize the importance of making such tests in order to determine how the treatment plant is operated. Then, again, there is no penalty enforced for not doing so, and the State Board of Health has neither the jurisdiction nor the funds to see that this work is done.

Another important feature of a sewage disposal plant is a measuring weir or other suitable measuring device for determining with a reasonable degree of accuracy the rate of flow of the sewage. Data on the sewage flow are valuable to a municipality from the standpoint of the operation not only of its disposal plant, but of its water-works system.

Engineers, however, have reason to be encouraged with the general results being obtained. As the various States place on their statutes laws regulating the operation of sewage treatment works, others will follow, and eventually the entire sewage practice with reference to operation and daily reports will be standardized.

TOWN PLANNING AND ITS RELATIONS TO THE PROFESSIONS INVOLVED

Discussion*

By Messrs. Edward M. Bassett, and J. E. Willoughby.

Edward M. Bassett,† Esq.—Mr. Adams‡ has referred to plans that he had helped to make and that he had seen others help to make which, after a number of years, were not carried out. One reason for this is because the elements that were carried out were not town planning; they were elements that had no security, and a change of management obliterated them, or the exploitation by private individuals, not regulated by law, allowed them to disappear.

A demonstration has been given of planning of the community and for the community, but there has been no demonstration of planning by the community. In the long run, communities are going to be the result of planning by the community. At present such planning is bad and incomplete. The powers of the community are not sufficiently cast into laws and made available so that planning by the community is effective. That gives all the greater place to those who will bring about, as many have helped to bring about, these demonstrations of private planning.

Some one will say, "Mr. Bassett is injecting his usual lawyer views into this subject", but this is not especially the position of the lawyer. Lawyers have no more to do with statutes than the architect, the engineer, the landscape man, or the ordinary citizen interested in good town planning; yet perhaps lawyers are a little more apt to come into the exact definition of these things, or make an effort to. The lawyer has no great place in town planning—he is the least of those who can really contribute; but possibly sometimes the lawyer can help to define, and if the definition is at all sound those who do the real work of town planning—the engineers, the architects, the landscape men, the social philosophers—will be better able to collaborate.

In nearly all discussions of town planning, various qualities or exhibitions of such planning are presented. What is town planning? By this is not meant the objects of town planning or the results of town planning. Town planning is the determination of the legal quality of land areas for public purposes. That is a simple and short statement that does not include the objects, or results, or the elements that constitute the subject, or describe what the different ones who do the town planning should contribute. To "deter-

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* Discussion of the paper by John Nolen, M. Am. Soc. C. E., continued from December, 1926, *Proceedings*.

† Director, Legal Div., Regional Plan of New York and Its Environs, New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 2049.

mine the legal qualities of land areas" requires the engineer, the architect, the landscape man, the social expert, and the housing expert, and the more they know about it the better they can do the town planning.

After twenty or thirty years, when the Town of Mariemont is turned over to the public authorities as a community to be run by the community—five or ten years after that date—what will be left of the town planning? It will be the legal quality of the land areas, namely, the streets, the parks, the sewer lines, the gas lines, the transit lines, the sites for public buildings—it will be those things that relate to the legal quality of the land. Nothing will be left regarding the type of house that has to be built when one burns down, except the zoning that has become fixed on the land, and that zoning is the impressing of a quality by law on the land. The color of the houses, the design of the houses, will all have disappeared because it cannot be expressed by valid law.

The laying out of the golf course in a park is landscape work, but it is not town planning. The fixing of the value of houses that must be constructed by private restrictions is very valuable in organization, but it is not town planning, and when the Town of Mariemont is turned over that will disappear. It may be that there are private restrictions that will continue in projects such as Palos Verdes and Mariemont. If those private restrictions are kept up, or the rule under private restrictions of a corporation that will dominate the community, then there is something that will, for a time, perpetuate itself, but it will not perpetuate itself forever—only for 15 to 30 years. In the long run communities are going to disregard those private restrictions, and that private control, unless all the land is owned by one corporation, will evaporate. In fifty years from now, the plan of Mariemont is going to be the quality that has been impressed on streets and parks, on waterways and water edges, on transit lines, and the zoning of private land; for proper town planning relates to private property as much as to public property because a legal quality can be impressed on private land.

If town planning is impressing the legal quality on land areas for public purposes, after that is done there is the town plan; the architect has his later work to do; the engineer has his later work to do in the street surfaces, the making of water-works and the gas and sewer systems; the landscape architect has his work to do in laying out the park surfaces and the location of civic centers. The man who combines all those qualities is the ideal city planner, but they all co-operate in carrying out the intention of the plan—and the plan is the impression of the legal quality of land areas for public purposes.

J. E. Willoughby,* M. Am. Soc. C. E. (by letter).†—Much of the town planning for American communities will be in the nature of a readjustment of existing features to conform to present and prospective conveniences and needs of the people and to provide for the co-ordination of those readjusted features to growth outward from the exterior boundaries of the city. Existing property rights will be affected. The individual loss sustained by the property owners as a result of the adjustment must be resolved into terms of

* Chf. Engr., A. C. L. R. R., Wilmington, N. C.

† Received by the Secretary, November 19, 1926.

money and paid for by the community. The failure of many experts engaged in town planning to recognize that obligation defeats the actual accomplishment of many worthy projects.

One of the more common lapses in this connection is the assumption that the steam and electric railways can be moved at the caprice of the planner and that the property owned and the rights enjoyed by those railways can be converted to public use without just compensation. The prosperity of all American commercial communities is dependent on rail transportation. Any plan that impairs the ability of the railways to serve best the needs, present and future, of the people is to be condemned as being contrary to the principles and ideals of town planning. The views of those engineers who are responsible for rail transport, as to what facilities and non-interferences by other forms of transport will be needed for the rail service, are worthy of consideration as a major requisite.

THE CINCINNATI CITY PLAN IS NOW LAW

Discussion*

BY MESSRS. ALFRED BETTMAN, EDWARD M. BASSETT, ARTHUR C. COMEY, E. P. GOODRICH, AND THOMAS T. TOWLES.

ALFRED BETTMAN,† ESQ.—Probably all are agreed that the city plan, however comprehensive, should have a considerable influence, even a considerable degree of control, in carrying itself out; that is, in actually becoming the design which is executed in the process of developing the city.

The problem of how to get these effects is not one of engineering in the technical sense, nor of city planning, nor of law. It is really a problem of political science, and the Ohio statute referred to in the paper, is the product of applying experience in practical politics to the problem of city planning and does not represent any fine-spun theory.

The basis of that statute is as follows: The regular officials of the City Administration as, for instance, the City Engineer, and the members of the legislative body, the City Councilmen, have an enormous mass of ordinary, every-day, pressing, urgent, routine problems. They are subjected to the political currents of the day—"political" not in any derogatory or sinister sense, but in the sense of vocal popular opinion or pressure of the day. Thus, the daily problems of the expenditure of public funds and the selections of public improvements take considerable of their time and energy and tend to be decided from the point of view of day-by-day pressures and day-by-day urgencies. The officials who have these daily problems are, therefore, by virtue of that fact, not the men in whom to repose the making of the city plan, which is a design for a long time in the future. Such a design therefore needs, as a matter of human nature, to be made by some body aloof from these daily urgencies which make it impossible to think in terms of long periods.

Consequently, a City Planning Commission should be created, with representation from the regular City Administration so that there may be coordination, but primarily aloof from it.

How shall the work of the Planning Commission, assuming it to be well done, be placed in a legal position actually to impress itself in results throughout the period of the plan? In order to think this problem out clearly, one must distinguish between that part of the plan which relates

* This discussion (of the paper by George B. Ford, Esq., presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926, and published in October, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† (Moulinier, Bettman & Hunt). Cincinnati, Ohio.

itself to the control of private property—that is, developments on private property, known as zoning—and that part which relates itself to the grand or master design for the location of permanent public improvements, such as streets, boulevards, markets, utilities, etc.

In so far as the plan is related to the control of the private owner's use of his own property, it obviously must be converted into law, and while naturally agreeing with Mr. Ford's praise of the Ohio System, the speaker has to disagree with his terminology. By "law", if it be used in any accurate sense, is meant a rule of conduct behind which there is a sanction enforced by the Courts. Therefore, this plan, which is a map and explanatory text, does not become law, as far as it controls the private use of the owner's property, until there is placed behind it some sanction enforced by the Courts.

That process of making law should not be reposed in any appointive administrative body, because it is of the very essence of the American constitutional system that law-making be reposed solely and exclusively in elected representatives of the people. Therefore, the converting of the zoning plan into law should be, and, in the Ohio statute, is, reposed in the City Council to be exercised by means of the passage of a zoning ordinance.

What effect should the Planning Commission's technically expert plan have on the text and the map of the zoning ordinance? The Ohio statute states that before the Council may depart from the zoning plan as made by the Planning Commission, it shall refer the departure back to the Commission, and if the Planning Commission does not agree to amend the plan, then a two-thirds vote of the Council is required to override the Planning Commission's opinion. That is something that is more than advisory and less than compulsory. It is a midway position. It does not differ, however, from many other examples in the statutes of every State in which the Council must act by a two-thirds vote. It is quite customary, for instance, to require a two-thirds or three-fourths vote of the membership of the legislative body to override the veto of the mayor, or governor, or president. Similarly, if the Planning Commission vetoes a departure from the plan, then approval by two-thirds of the Council, instead of the ordinary majority, is necessary to convert that plan into the sanctions and the methods of enforcement which are characteristic of law.

How about that part of the plan which relates to public improvements—where the future streets shall be; where the boulevard system shall be extended; where the railroad terminals shall be; and all the other features of this grand master design which are to be carried into effect throughout the years of a generation or more? This is a point not raised by Mr. Ford, but it is a point which is involved in the process of drafting a city planning statute. It seems quite obvious that the City Council of any particular day or month or year should not be given the power to approve or disapprove the whole initial master design or city plan, because, by virtue of the method of choosing members of the Council, its procedure, the technical facilities at its disposal, and its subjection to all the current pressures

of the day, it has not the qualifications, the equipment, or the motives which are appropriate to making a grand design of future public improvements running through a long period of years. Therefore, the making of such a design cannot appropriately be left to a City Council, but needs to be in this more aloof type of body known as the City Planning Commission, assisted by technical skill.

What effect, however, shall this grand design have on the actual location of the future public improvements? From year to year a City Council has the problem of raising sufficient monies and expending them for the things that are to be built that year. In other words, it appropriates the money, whether it be tax funds, bond funds, or other funds. Whichever type of fund may be expended, it is again fundamental in the American political system that in the elected representatives of the people constituting the legislative body shall the power of the actual expenditure of public money be reposed, and that there ought to be no power in any planning commission, or any administrative board, or any appointed officials to decide finally the expenditure of any particular sum of money on any particular type of improvement at any particular time; and the Ohio statute does not attempt to give any such power to the Planning Commission. All that it attempts to do in the case of future public improvements is exactly that which the speaker has attempted to describe in relation to the zoning ordinance—it is something more than merely advisory and something less than compulsory, namely, that before Council may locate, by its own expenditure of public monies, or by a franchise, a public improvement or utility other than at the place designated in the plan, it shall first have to refer this proposed departure from the plan to the Planning Commission, and if it does not succeed in convincing the Commission to amend the plan accordingly, then a two-thirds vote of the Council is requisite to override the plan and to locate the improvement of the utility in violation of it.

That is quite different, at least in the terminology of lawyers, from making the city plan law. That is, however, it seems to the speaker, giving a sensible amount of force to the city plan, to assure that the plan cannot be carelessly overridden. The plan may not be simply ignored or forgotten. Council must take it into account to the extent of having the fact that the plan is there called to its attention, to the extent of having to go to the Planning Commission with any proposal that violates or departs from the plan, and to the extent that more than a bare majority of the Council is needed to depart from the plan. This is simply keeping the plan alive and with some influence, in the ordinary routine process of legislating from day to day in the expenditure of public funds. That is the theory of the Ohio legislation—not theoretical in any sense which is contrasted with the practical, but the theory of the applied politics of it.

The Cincinnati Plan was officially adopted in May, 1925. It is a little soon to draw any firm conclusions as to how it will turn out, but it looks as if the theory will succeed and as if the plan will be given some strength, more than a mere negligible piece of advice.

For instance, shortly after the plan was adopted, the City Engineer sent to the Planning Commission certain detailed plans and specifications. These called for the re-surfacing of certain streets at their then width, whereas the City Plan called for a widening of these roadways with a corresponding narrowing of the sidewalks. The streets were in bad condition; there was tremendous pressure to have them re-surfaced immediately. The narrowing of the sidewalks would deprive the property owners of the use they were making of that space. They had machinery beneath the sidewalks and areaways, and entrances to basement restaurants, etc. Therefore, the City Plan disturbed their free use of city property and they opposed it in that respect. Under the urgency of re-surfacing and the private free use that was being made of the sidewalk space, Council passed the ordinances in accordance with the plans and specifications without paying any attention to the general plan. This was not called to its attention before it passed these ordinances. Thereupon, the City Solicitor notified Council that the ordinances could not take effect, and that he would enjoin the re-surfacing of the street at the original width unless the ordinances were first submitted to the Planning Commission.

This power given to the plan, something more than advisory, forced a reconsideration. The Planning Commission took the matter under consideration, and the private property owners subjected the Commission to the same pressure. The Planning Commission, however, was not as amenable to that kind of pressure as the Council and refused to depart from the plan, and that is where the matter now stands—the Planning Commission refuses to depart from the plan and the plan has sufficient popular prestige so that Council does not feel like overriding the Planning Commission.

The city owns a piece of property that was the former site of a hospital. There has been a movement on foot to sell it as it is very valuable. The plan designates this site as the location of the future Public Auditorium. Thus far, the plan is winning; and it looks as if it will win in the end.

Similarly, there is a movement, on the part of the Cincinnati Baseball Association, to acquire another piece of property owned by the city for a new baseball field, which piece of property the plan designates as the location of the public City Athletic Field. The Baseball Association finds itself under the necessity not merely of creating a favorable sentiment in Council—and it would be very easy to do that because baseball is the most popular sport in the city—but also of convincing the Planning Commission. At least the City Council will not act until the Planning Commission has had a good opportunity either to amend the plan or stick by it in that particular.

It may be seen, therefore, that ease of departure from the plan is much lessened by giving it the legal status conferred by the Ohio statute which Mr. Ford has described.

EDWARD M. BASSETT,* ESQ.—The speaker has given considerable study to this very interesting paper of Mr. Ford. His point of view is almost the same

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as those of the other discussors, the only difference being as to where to place the emphasis. One of the discussors plainly would put the emphasis on public opinion, and he may be right; but the powers of the community have not been crystallized by public opinion so as to bring about better town planning. The speaker would probably give emphasis to this crystallization of community powers into laws that will bring results.

Officials and citizens of outside towns ask, "How can we prevent the issue of building permits on misplaced private streets?" The answer is, "You cannot prevent it. There is no statute in the State of New York under which you can prevent it, and no ordinance of your village under which you can prevent it." They ask, "How can we prevent a lane being laid out that upsets the plan of the village?" and they can hardly believe it when they are told that they cannot prevent it, that "you can spend some money if you want for fighting in Court, but you are going to be beaten because the community power in that respect has not been put where you can avail yourself of it."

Many have become convinced that a planning commission must be more than merely advisory. It is now well recognized that a State Legislature can provide for the appointment of a body to administer a subject. The legislative body prescribes the rule of conduct or the limits within which that administrative body may act on details. The advantage in this method is that although a legislative act cannot be modified by the Court, the doings of an administrative body, acting under a rule duly prescribed, can be reviewed by the Court and, if necessary, sent back to be done again. Therefore, there is great elasticity and adaptability in a functioning administrative body such as many believe planning commissions can be.

Two of the most highly organized laws for city planning in the United States are those of Cincinnati, Ohio, and New York, N. Y. The City of New York is not always thought of as highly organized in its procedure, but through long periods of years there has been built up a wonderfully good procedure. In some respects, the City of Cincinnati has improved on it.

The speaker would like to compare some of these improvements by a sort of parallel column method. New York City has an official city map or plan. It covers, however, only streets and parks. It will be noticed that it covers only those land areas which are the subject of dedication. The first step in a taking is the placing of an area for a street, open place, or park on the official city map. The next step is the draft damage map, which finds its data on the official city map. Consequently, the official city map must be precise.

In Cincinnati the official plan of the city shows streets, parks, and open places, but it also shows sites for public buildings, locations of public utilities, and all kinds of city planning projects. The real difference is that Cincinnati includes in its official plan those things which are not the subject of dedication. In that particular respect there is a permanency, or there ought to be a permanency, in the New York official map which one can hardly expect if the same control applies to public building sites and public utilities. Changes of mapped streets and parks should be made more difficult of accomplishment than changes of grades, locations of proposed public utilities, or proposed sites for public buildings.

Next, New York City has no planning commission, but every amendment to the official city map is reported on by the Chief Engineer of the Board of Estimate and Apportionment. Therefore, it is the legislative authority advised by the Chief Engineer that makes changes of the official city map possible. The City of Cincinnati, however, has a planning commission which is assisted by the City Engineer and its decisions should carry a great deal of weight. The reports of the late Nelson P. Lewis, M. Am. Soc. C. E., and, later, of Arthur S. Tuttle, M. Am. Soc. C. E., the only Chief Engineers of the Board of Estimate and Apportionment of Greater New York since 1904, have been as sound and disinterested as those of any commission could be.

Next, in New York, plats must be approved by the Board of Estimate and Apportionment, or by the Chief Engineer before they can be placed on record. In Cincinnati, they must be approved by the Commission before they can be put on record.

Then, in New York, under State law, it is a misdemeanor to make a deed of land before the plat is filed, if it is land in a sub-division. This is also true of the Cincinnati law. In New York, the law is a dead letter and will probably turn out to be the same in Cincinnati.

There are certain shortcomings in these two plans, however, and the speaker will use both cities to show how wide these shortcomings are. In either city private streets can be laid out by any citizen who chooses to do so on his own land, without any power on the part of the city officials to stop it. They can refuse to make it a public street, but the man who owns his own land can lay out a private street. Next, the man who lays out those private streets can ask for permits to put up buildings on them and the authorities in either New York or Cincinnati cannot lawfully refuse the building permit. A man, in either city, may get a permit on a private street, which may be entirely in conflict with the city plan, and when twenty or thirty houses have been built on a private street, it is only one step to persuade the authorities to put in the sewer, lighting, etc. They cannot very well tear down the houses, and they make the best of a bad situation. The law in all cities is defective in that it allows private streets to be laid out contrary to the plan and permits houses to be built as a matter of legal right on any misplaced private street.

Next, in every State, except Pennsylvania, as far as the speaker knows, an applicant for a permit to construct a building within the limits of a mapped street not yet opened, can obtain it.

Again, small parks, especially small parks for playgrounds, are to-day just about as much of a need in a big modern city as a street. Sooner or later the taxpayers on the surrounding land have to pay for them. Sometimes buildings are torn down and the entire cost of land and buildings is assessed. There should be a method of obtaining small parks for playgrounds as automatically as streets. This should be before the land is built over. One is nearly as important as the other, but because the community can exist without playgrounds and it cannot exist without streets, new streets are required, but playgrounds can go until the land is built up and then the houses must be pulled down at great expense to create the playground.

There ought to be, in both New York and Cincinnati, a more intimate method of co-operation between the land-owners and the officials in laying out secondary streets. Many mistakes have been made in New York because there is a hard and fast official city plan before the streets are opened, to which the land-owners must conform. Sometimes, bluffs have had to be torn down, which had better been left standing. Sometimes, there has not been any conformity with the plainest needs of the environment. Yet the gridiron plan has been so inflexible that there was nothing to do but adhere to it. There are drawbacks to having a fixed plan to which the private owner must absolutely conform. He ought to have a chance, at least, to talk it over with an advisory administrative planning commission.

There ought to be an intimate method of co-operation between the property owner and the officials in the application of zoning. A small business center is needed, and a buffer district for apartment houses and a place for detached houses outside is often needed. Too great fixity prevents that intimate co-operation between landowners and the officials that ought to exist if a rational city planning method is to be brought about. Planning commissions might properly have power regarding details. These shortcomings in great cities are well worth considering in progressive study of city planning.

Some claim that private land is, and that public land is not, the subject of zoning. By and large this statement carries, but that is not the line of distinction. Zoning relates to buildable land, whether it is private or public. In New York, if the Park Commissioner wants to erect a building in a park, it must conform to the height and area provisions because all the park land is zoned. The streets, of course, are not buildable land, and they cannot be zoned, but a site for a public school or firehouse is zoned, and the public authority must conform to the zoning plan just as much as if the land was privately owned.

ARTHUR C. COMEY,* M. AM. SOC. C. E.—Mr. Ford has given three questions to be answered, each of which relates to at least three distinct aspects of the subject of city planning powers. It seems that the only way to get intelligent answers is to split them up accordingly.

The powers under discussion are exercised over three main categories of property: (1) Private property, which under the plan is to remain private but controlled to a degree by zoning, building laws, and similar applications of the "police power", so-called, for the general good; (2) public property, either already existing or to be acquired substantially at once or later by a single act of acquisition, for streets, parks, public buildings, etc., under the power of eminent domain, with compensation for those damaged; and (3) private property which later is to become public and, therefore, needs to be controlled so as to fit in with the city plan when it ultimately is absorbed, this control being at present, in the United States outside of Pennsylvania at least, largely in the stage of proposals either to restrict under the police power permits for private buildings in the beds of mapped streets and other areas, or to take away by eminent domain the right for damages to such

* Cons. City Planning Engr., Cambridge, Mass.

buildings when later taken as part of the public acquisition—the latter method being proposed in Massachusetts* and preferred by the "Hoover" Committee on City Planning of the U. S. Department of Commerce.†

General theories and "fundamental" principles need not give undue concern as the former must meet the actual needs or fall and the latter are apt to be merely substitutes for thinking. Any of the opposing points of view in regard to the questions under discussion can be supported by a slight and often unconscious twisting of the same words and phrases. For example, one is told that legislative powers should not be delegated from the city council and then finds delegation justified by labeling the powers as administrative. Some regard, however, should be paid to practice and its evolution and to the ease of application of proposals. Many an otherwise meritorious plan has failed because it is unnecessarily counter to current habits or because it is too cumbersome or involved in procedure.

The main questions of city planning powers, therefore, as applied to each of the three categories previously listed, are:

I.—Shall the city have city planning powers?

II.—Shall any body other than the city council exercise any control?

III.—If so, shall the city plan commission exercise any control?

In regard to Question I, there seems to be unanimity that considerable city planning powers should exist (1) to zone private property, enforce building laws, etc., as at present effective in most States; (2) to carry out a city plan by acquisition of lands needed for streets, parks, etc., also in large degree now in effect in the United States; and (3) to protect the beds of mapped streets and other areas, as yet only proposed.

In regard to Questions II and III, Mr. Ford urges that the city plan commission should have extensive powers to enforce the city plan it adopts, particularly as regards the second and third categories of property, and adduces valid reasons in support of his position. Yet he, as well as the "Hoover" Committee and others, apparently is satisfied that with regard to the first category the opposite course should be pursued. Zoning is at present practically universally enacted and from time to time amended by the city council, utilizing in so far as it sees fit the advice of the city plan commission or a similar advisory body. This accords with the habit and trend of local government to-day, which seeks to centralize control of the larger matters of vital import in the elected body, the city council, thereby dignifying that body and avoiding conflicts that are inevitable where there are two co-ordinate bodies. It would seem that the same method might work as well for other aspects of the city plan. It has not been proven that it does not and, therefore, a logical first step in the evolution of city planning powers would appear to be to give the city council the power to adopt the city plan, and not delegate these powers to any other body. Where this action proves ineffective local option may well permit the electorate, or the council itself, to delegate powers.

* *Bulletin 16*, Massachusetts Federation of Planning Boards; also, House Bill No. 504, Mass. General Court of 1926.

† "A Proposed Standard City Planning Enabling Act."

Even if the answer to Question II is in favor of delegation of power, there are arguments against giving it to planning commissions. Such bodies are preferably composed chiefly of high-minded citizens donating their services. They should be free of administrative detail, and moreover the very fact that they are advisory only assures them greater recognition from all quarters. The arrangement in Rochester, N. Y., with a Department of City Planning under a single head and an advisory commission, appears to be a practical means of securing both the advantages of a disinterested commission and an efficient administrative agency.

In brief, there is no one royal road to successful carrying out of city plans. In Massachusetts, for example, the effort to apply Mr. Ford's proposals to-day would probably wreck the entire city planning structure throughout the State. Powers to planning boards would be a red rag in the Legislature, as well as in many cities and towns where these new boards, at first viewed with suspicion, have gradually acquired general confidence, based very largely on the fact of their advisory character. These boards meet as a rule once a month. Were their sessions to any large degree taken up by executive detail, such as approving plats, their ability to deal with the larger aspects of the city plan would be apt to be seriously impaired. It may be argued that they should have staffs capable of handling all this detail. In that case the Rochester method would seem preferable, clothing the administrative head actually responsible with the power and leaving the commission as advisory.

In Massachusetts few people at present would see much need or reason for departing from good and tested practice until it has been tried out as applied to city planning. It has not been tried out yet because thus far the more vital parts of such city planning powers as those under discussion have not been delegated to the municipality at all. There is no adequate power in Massachusetts communities to enforce a city plan. That power might well first be lodged in the legislative body, then, if this is not successful, that, in itself, will be an effective argument for transferring it to an administrative body. That is exactly the proposed procedure in Massachusetts at present in the matter of protecting mapped streets, the third category noted.

Hitherto in Massachusetts it has seemed worth while to try the established practice of having one legislative body handle everything it possibly can. There, as in most States, the city council adopts the zoning law (except in Boston, where zoning and building laws are State acts). The planning board or other advisory agency draws up a zoning plan and transmits it to the council. Sometimes the council tinkers with it, but more often adopts it intact. In practice this procedure seems to work out. The fact that zoning directly controls private property does not appear to be a fundamental reason why the zoning law should be entirely controlled by the council any more than other parts of city planning, also affecting private property.

In conclusion, it seems generally agreed that more powers are needed in practically every State, Pennsylvania possibly excepted, for enforcing a city plan. Whether such powers are exercised by the city council or by an administrative body may well be left to each municipality, either by the

electorate accepting this feature of an enabling act or charter provision, or by the council establishing the procedure by ordinance.

The city plan commission may well be charged with the duty of preparing a complete city plan for adoption by the city council. The control of plats and other administrative details may well be lodged in a single-headed city planning department working in conformity to the city plan. As safeguards for comprehensive planning the council's action on all matters affecting the city plan may properly be required to be held up for a reasonable period pending the city plan commission's report thereon. It may even be required that any action other than conformity to the commission's report shall require special publicity and a three-fourths vote of the entire council.

E. P. Goodrich,* M. Am. Soc. C. E.—Based on a rather wide range of personal experience, the speaker takes the liberty of crystallizing the thought on the question here discussed. In the first place there must be as rigid a condition as is possible, with as much power as the public will permit in the hands of those who are forming and administering the city plan. At the same time the plan must be as elastic as possible. Those two things seem antithetical, but there must be created by some device, some means of adapting a plan to the needs when something new arises, through a process of evolution. In the original grant of power, of course, one cannot go further than public opinion at that time will uphold. Sometimes, it is possible, by looking into the future, to "put something over" on the public. Luckily, the ones who normally are charged with the formulation of plans are usually looking out for the public interests.

At the same time that a plan is being created and carried out, public opinion must be formed. Sometimes that can be done by the city plan commission, or a similar body, but experience shows that the best method is by an independent citizen body or city movement which will keep behind the public officials and force them to create the best possible law or plan at the time and for as far into the future as possible, and then force them to administer it to the best interest of all the community.

This need of an independent citizen body constantly behind the plan, constantly at the elbow of the officials, even if the latter be the highest class of citizenry, is imperative to get the best results. The speaker thinks he is free to quote Mr. Bettman who stated that even in Cincinnati a citizen body will be necessary continuously to see that the city plan is carried through and maintained.

Thomas T. Towles,† M. Am. Soc. C. E.—Although the speaker has been greatly interested in Mr. Ford's stimulating paper he differs very decidedly from his conclusions. There seems to be an erroneous notion that the city plan could at any time be a fixed conception to which the city's growth must adhere. If the people or the authorities of the city have the idea (and at some time they will entertain such a notion) that a formal plan very difficult of change is imposed on them from the outside, so to speak, it would seem to

* Cons. Engr., New York, N. Y.

† Design Engr., Div. of Eng. and Constr., Stone & Webster, Inc., Boston, Mass.

be almost impossible of effective enforcement. Such would be particularly the case with the smaller cities or towns in which the interest and the influence of the individual is more active and direct than is the case in the large centers of population.

The comprehensive plan of Cincinnati involves not only the details of street layout, but also the particular layouts of parks and playgrounds and even the plans for all public utilities (perhaps, however, general in nature in this case). The speaker's experience has been that if the plan went into all details of this nature the authorities of the town would have to ask the city council for a change in it, every month. A few repetitions of this would simply put the plan in discard. A community is subject to constant change not simply as regards population, but to change and development as regards its ideas, general outlook, and even its necessities. The city plan must be reasonably capable of taking new form, elastic in its nature, and responsive to the needs and changing conceptions of the community.

The necessity of having a body of citizens outside the constituted authorities of the city earnestly devoted to the problems of city planning has also been discussed. The City Plan Commission of Richmond, Va., is composed of five public officials, the mayor, and the directors of the several departments of the city government. This body of officials simply does not act in any sense as a city planning commission. Each official is concerned almost entirely with the routine duties of his office. Furthermore, it is very seldom that a public official conveys to the people the notion of disinterestedness that would be attached to a commission presumably uninfluenced by political considerations.

Sufficient stress has not been given to the importance of public opinion. To obtain a thorough consistent execution of any largely conceived plan, it is necessary to educate public opinion to some conception of the importance of planning for the community as a whole. Furthermore, it seems fundamental that a plan should be, in some real sense at least, a result of the expressed needs and a product of the will of the community.

THE NEW YORK STATE BARGE CANAL AND ITS OPERATION

Discussion*

BY MESSRS. HARRY TAYLOR, E. P. GOODRICH, F. LAVIS, J. K. FINCH, AND E. E. KING.

HARRY TAYLOR,† M. AM. SOC. C. E.—It seems that the people of New York State and throughout the whole country have not really awakened to the value of the Barge Canal. They speak of it as being more or less of a failure, as Mr. Finch has stated. The entire Middle West seems to be "sold" with the idea that a deep-water canal, either down the St. Lawrence River or across the State of New York, will solve all their troubles. They speak of tremendously cheap transportation on the Great Lakes and have an idea that this will be extended to the final port to which the grain is carried. Transportation on the Great Lakes is, presumably, the cheapest transportation in the world. There are a number of reasons for that; for example, the wonderful terminal facilities and the fact that after the boats start in the spring they make practically continuous voyages until they are laid up in the fall. There is practically no delay at the terminals.

The lake boats are also built, per ton of carrying capacity, very much cheaper than ocean steamers. Boats suitable for the Barge Canal can be built for less than lake boats and operated more cheaply. Why should not the lake boat bring commerce from the Upper Lakes to the Eastern ports at Buffalo, or better, at Oswego, transfer there to the barge, carry it across the State of New York, and there transfer it to the ocean steamer, either at Albany or New York. In that way the tonnage is carried in each boat by the cheapest possible carrier and the transfer charges would be more than saved.

E. P. GOODRICH,‡ M. AM. SOC. C. E.—The author has pointed out with absolute accuracy that one of the principal handicaps under which the Barge Canal is now operating is the lack of boats. In this connection, it is apparent that the development of a port at Albany, where freight transfer can be made, will make it possible to shorten the total Barge Canal trip from Buffalo to New York. For many commodities this will be feasible, grain, for example, provided a grain elevator is established at the Albany port, as is required by the Board of Army Engineers. Under such circumstances, about 50% additional service can be secured from the existing boats so that the development of a port at Albany will be equivalent to that increase in the possibilities of Barge Canal operation.

* Discussion on the paper by Roy G. Finch, M. Am. Soc. C. E., continued from December, 1926, *Proceedings*.

† Maj.-Gen., U. S. Army (*Retired*), Washington, D. C.

‡ Cons. Engr., New York, N. Y.

F. LAVIS,* M. AM. SOC. C. E. (by letter).†—The author states that "there has never been a time when there was a more crying need for clear thinking in relation to waterways than at present." The writer agrees that there is certainly need of clear thinking at this time, but submits that there was just as much need twenty years ago, before the $175 000 000 which the Barge Canal cost was expended, and that clear thinking then would not only have saved this large sum for construction, but also the many millions more it has since cost in annual deficits.

Reference to the files of *Engineering News* for the period during which the then proposed bond issue was being discussed, will show that there were at least a few people, even at that time, who could think clearly and along lines of sound economics in regard to this matter, but their's were "voices crying in the wilderness", to which no heed was given. Let us therefore, as Mr. Finch advocates, try and think clearly now before any more money is wasted on this or the proposed ship canal.

One of the principal difficulties in getting this clear thinking in considering waterways and water-power problems is the ingrained idea that is firmly rooted in the minds of probably 90% of the people of the United States that water is something provided by a Divine Providence and that man has only to stretch out his hand and take and use it as a free gift. Laymen, and even many engineers, ignore, or are unaware of the fact, that the development of electrical energy by water power and its transmission to the place where it may be used is often quite as expensive, sometimes more so, than the development of power by the combustion of fuel. So, also, do they think of waterways, ignoring the fact that artificial waterways often cost more to build and operate than railways or highways having equal capacity for traffic and capable of providing a far more flexible service. Unlike the canal-boat, a railroad car does not merely travel from a point on the main line of its home railroad to another point on that same main line, but it can and does travel from any side track at any point of production—mine, forest, or factory—to any other side track where the material it carries is needed. The writer has no knowledge of the relative cost of the actual power required to haul a ton on rails as compared with a ton on the water, but it hardly seems likely that the cost is less on the water than on the railroads.

Mr. Finch states that one of the reasons why the canal has not been used to a greater extent is because there has not been and even now is not an adequate supply of boats. He draws or attempts to draw, an analogy between this and a railroad without rolling stock. A railroad at the start provides rolling stock for the business in sight and increases its rolling stock as the business develops, and only as it develops. The fact that after eight years of operation, the demand for traffic on the Barge Canal is only about 10% of its capacity is surely evidence that there is no crying need for this adjunct to the State's transportation facilities. It seems inconceivable, if transportation by this means was desirable, was economically sound, was a convenience, or was, on the whole, cheaper, that the many commercial enterprises in this

* Cons. Engr., Jersey City, N. J.

† Received by the Secretary, October 29, 1926.

country, which are ever on the alert to save a few cents, to say nothing of dollars, would not have availed themselves long ago of the opportunity which the canal is supposed to offer.

Even with the canal waterway offered free to users, the State carrying all the expense of maintenance and operation as well as interest and amortization on the original investment, its value and utility is so little apparent that its use is really negligible, and its importance as a factor in transportation almost nil.

Maurice W. Williams, M. Am. Soc. C. E., has recently stated that by 1939 the annual losses on the Barge Canal for maintenance, interest, and amortization will have reached about \$150 000 000, in addition to the first cost of \$175 000 000. Adding to this interest at 4% per annum on the losses for one-half the period, 1918 to 1939, there is a further loss to the community of \$55 000 000, or by 1939 a total of \$380 000 000.

The argument is advanced that the losses on the Barge Canal are offset by savings to shippers by reason of the lower freight rates on the railroads which are maintained by reason of the water competition. Of course, the fallacy of this is evident not only because freight rates on railroads are pretty closely regulated by the Interstate Commerce Commission, but because, in any event, looking at the matter from the broad standpoint of the whole National or even State economy, and in view of the fact that the railroads are owned by the public and by the shippers, savings or expenditures on them are for account of the people as a whole.

The investment, therefore, of \$175 000 000 and annual expenditures totaling \$205 000 000 more of the people's money to save money for certain shippers at the expense of the people who have invested in the railroads, hardly seems sound economy.

In the early days of canals, when the railroads were operated as unregulated and almost unrestricted monopolies, there might have been some force in an argument of savings by reason of the water competition, but it is not sound in view of the conditions under which the railroads are operated to-day. It is wise therefore, as Mr. Finch suggests, to try and think clearly in regard to the economics of this question. This requires some consideration of the economics of transportation, the principal factor in which is the railroad system of this country. It should be remembered that the railroads to-day are under strict and, on the whole, quite efficient regulation by the National Government, as well as by the individual States.

The cost of transportation by the railroads is now based closely on actual costs of operation and maintenance, plus a very moderate rate of interest (6%) on the investment in the property. The amount of this investment is also fixed by the Government and represents a sum which is undoubtedly not greater than the reproduction cost of the properties as of the time the various parts were constructed, and less than their reproduction cost to-day.

On the whole, the money invested in the railroads is the money of the people and, in the long run, any gain or loss is a gain or loss to the nation as as a whole. Transportation by railroad is so vital to the continuing life

of the country, to say nothing of its continuing prosperity, that the railroads must be kept at a high state of efficiency if the nation is to continue to live and prosper.

There is of course no argument to be advanced against the development of waterways *per se*, but when they are developed, as was the Barge Canal, solely on the basis of offering competition to the railroads, and when this competition results in a loss to the people of the State of more than \$370 000 000, such expenditures are to be deplored.

The construction of any artificial waterway can usually be justified only if, after taking into consideration annual charges for interest and amortization on the investment, costs of maintenance and operation of the waterway, and costs of operation of the vehicles traveling over it, it produces ton-miles at rates comparable with or less than rates at which these same or similar ton-miles can be produced by other means. This, as far as the writer knows, has not yet been accomplished on any inland waterway in the United States, the Great Lakes not being included, of course.

One trouble with clear thinking in this matter is that so many people are still living and thinking in the manner of the days when the waterways were as vital factors in the life of the nation as railways and highways are to-day. They have not yet realized that because of extreme flexibility as to location, time, and character, and because of adaptability of vehicles, the highways and railways are rendering services so far superior to those which these inland waterways can possibly render that, except in a very few instances, these latter have little place in the modern plan of economic transportation.

J. K. Finch,* Assoc. M. Am. Soc. C. E. (by letter).†—This clear brief statement of the problem of the New York State Barge Canal by the State Engineer and Surveyor brings before the Society for discussion a problem in engineering economics of the first magnitude.

In the writer's opinion too many great public undertakings are settled pro or con by popular appeals based on pride, patriotism, or politics. The public is too often called on to decide questions which actually demand expert and unbiased study by economic and engineering experts. Kipling's village decided by popular vote that the world is flat. Has not this been done with the Barge Canal?

In many cases engineers themselves have carefully avoided the economic issues involved and have been content to act simply as agents in carrying out the structural requirements of undertakings which have been decided on by others. One of the greatest engineers, Wellington, pointed out clearly that the engineer's duties went beyond mere design and required him to advise his client whether the project in general was economically sound. Indeed, the economic factors of most great engineering problems are so intimately tied up with the technical engineering possibilities and questions involved, that a man must have engineering training as well as a thorough understanding of economic principles to handle them.

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† Received by the Secretary, November 9, 1926.

Engineering schools have been very backward in recognizing this fact. In general, they have devoted their energies to perfecting their students in the latest advances in science and their applications, and developments in the technique of engineering. This has been one of their fundamental duties; but there can be no question that the reason why engineers have been adverse to giving attention to the broader economic problems of their professional work is due to the fact that they realize their lack of the training and background necessary to handle such questions. The writer ventures to express the opinion that the engineering schools and the professional societies should turn their attention to this field and that in this phase of their activities modern engineers can be of still greater usefulness to their country and their clients. From this standpoint the paper in question is an important contribution to engineering activities and merits the fullest pôssible discussion.

Mr. Finch has outlined the history of the Barge Canal. Briefly, it shows that for fifty years "Clinton's ditch" was a great economic success. In the beginning it had practically no competition, and for a time it competed successfully with the young railroads, which, however, cut more and more into its traffic. Then came the Civil War, and, with the closing of the Mississippi, both the canal and the railroads had all that they could handle. By 1872, however, the decline in the canal business began. The Erie Canal went through the same experience as most of the other canals of the country. Like the others, the Erie had its day of great economic usefulness, but, due to the fact that canals were slow means of moving freight, that they were only available during part of the year on account of ice conditions, and in general, due to the expansion of the country, they became only parts of through systems and therefore costly transshipment was necessary. They could not deliver freight to the consumer's door as was often possible through railroad sidings, and because of the increasing efficiency and effectiveness of railroad operation, the Erie, like other canals, became practically an abandoned waterway.

At an early date in its history it had been saddled with so-called feeder canals which proved to be financial burdens. At various times it was widened and deepened and, until its last years, these efforts at increasing its usefulness had been productive investments. Therefore, in spite of the fact that 3 000 of the 3 750 miles of canals built in the United States had been abandoned—because, like the Erie, they had outlived their usefulness—an agitation was started in the Nineties to again widen and deepen the Erie, and make it into a Barge Canal, with the idea that this would again prolong its life.

In 1903 "the people of the State of New York" decided to undertake this work, which has cost them to date more than \$178 000 000 (omitting interest and maintenance charges which are not earned and amount to about \$7 000 000 per year). The result of this expert opinion on the part of the public has been on a par with that of Kipling's village already mentioned. The Barge Canal now carries about one-tenth of the traffic it could handle, and it has been estimated* that it would cost "the people of the State of New

* See Report of the Bureau of Railroad Economics, Washington, D. C., June, 1925.

York" less to pay the railroads to handle this traffic than it does for them to operate the canal.

The arguments advanced in 1903 urging this expenditure were based largely on appeals to the pride of "the people of New York State." The Erie Canal made the Empire State; the Barge Canal will retain for this State its economic supremacy. More than 70% of the population of New York, that is 7% of the population of the United States, live within a 30-min. walk of the canal. This means that the products of 8 000 000 people need and should have a cheap means of transportation, such as the Barge Canal will afford, etc. These, of course, are mere statements and not the kind of sound economic truths that can be backed up with operating results.

It was obvious that traffic originating within the State could not be expected to be sufficient in volume to justify the canal. The failure of the feeder system of earlier years showed this. The population along the canal, therefore, is of no particular interest. Mr. A. Barton Hepburn, however, has clearly stated the principal source of traffic hoped for, namely, Western grain. In a book,* primarily devoted to an attempt to justify the Barge Canal enlargement, he argued for several pages that the Barge Canal would bring back to New York the grain export business which was rapidly being taken away from it by Canadian facilities, thus giving the City of Montreal the export supremacy formerly held by New York City. Later, in one paragraph, he demolishes this argument with the statement "the growth of our population has shattered this hope, for it is likely in the future we will consume at home practically all the grain we raise." Typical of the spirit of the arguments advanced at that time, he goes on, "our enlarged waterway must and will find other means of justifying its existence." He does not state what they are and "the people" are still looking for them.

A most thorough study of the inland waterway problem was made by Mr. Charles Whiting Baker in 1920.† He concluded that most of the inland waterways had served their purpose and could not compete with modern means of transportation. These studies are worthy of careful attention in connection with this problem.

At present, therefore, "the people of New York State" are saddled with the biggest "white elephant" ever purchased, the most colossal engineering failure of modern times, and the question is what to do about it. An attempt has been made to turn it over to the Federal Government as a possible basis for a ship canal. The War Department engineers apparently do not take to this idea. Mr. Finch now brings this problem before the Society. He suggests that the State has not done all that it should to make the Barge Canal useful. A few years ago it lacked terminal facilities, but in spite of the fact that "terminals have been constructed at every city and nearly every village along the line of the canal," traffic goes elsewhere.

It is now stated that one of the reasons why the canal has failed to get traffic is the lack of boats for transporting this traffic. It is suggested that

* "Artificial Waterways of the World", N. Y., McMillan, 1914.

† "The Future of Inland Water Transportation," a series of articles in *Engineering News-Record*, Vol. 84.

the canal be again widened and deepened and that the State go into the navigation business, either directly or by financial aid. While it is true that the Barge Canal has not had all possible chances to make it a success, is it not also true that if the traffic was there to justify a larger canal equipment there would be private capital ready and anxious to go into the canal transportation business?

In his discussion of the problem* Mr. Fay endeavors to excuse the canal for its failure to produce; he argues that Canada has and is spending millions on her waterways, hopes that Americans may be wide awake and get some of the grain export trade from Canada; he foretells a great future when both canals and railroads will be needed to handle the huge volume of traffic, and concludes that "the people of the State of New York have builded wiser than they knew in providing the modern Barge Canal." This is simply an opinion and does not seem to be backed up by facts.

Barge Canal traffic must come from two sources: (*a*) within the State; that is, local traffic along the line; or (*b*) through traffic. While the writer does not pose as an expert in these matters he would like to ask three questions.

1.—Is it not true that local traffic can never be expected to justify the New York State Barge Canal?

2.—Is it not true that the only type of through traffic in sight and in sufficient volume to justify the canal is the export grain business?

3.—Is it reasonable to suppose that the New York State Barge Canal, merely a possible link in a through route and hence involving transshipment, can successfully compete with Canadian through routes in handling export grain, which admittedly originates almost entirely in Canadian territory?

There are sound economic reasons why New York is losing her grain business. The writer knows of no sound reasons why the canal will be, or can be made in the near future, an economic success.

It has been argued that the Barge Canal is needed to keep railroad rates down to reasonable figures. This is equivalent to stating that the commission form of control for privately owned public utilities is a failure. It assumes that effective regulation of transportation requires the construction of competing lines at huge costs, lines which would do little or no business but are necessary to protect the public from exorbitant charges. Such a policy would, of course, result in economic disaster. Government ownership would be preferable to this, and experiments to date along these lines show conclusively that in most cases such ownership is simply another name for political mismanagement which, unlike private management, cannot be controlled and is both dangerous and costly.

Is it not best to reduce the maintenance costs of the Barge Canal to a minimum until a time arrives when traffic requirements can be based on something better than "hope"; to put capital cost down to profit and loss; and to look on the New York State Barge Canal as a great lesson purchased by "the people of New York State" at no less than one-half the cost of the Panama

* *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1884.

Canal, for the purpose of proving that great engineering undertakings are only successful when based on sound economic reasoning and advice.

E. E. KING,* ASSOC. M. AM. SOC. C. E. (by letter).†—In comparing costs by different systems of transportation, it is necessary to consider in any shipment the cost from the door of the shipper to the door of the consignee. In the case of small packages shipped by freight there is probably little difference in the cost of transportation whether the shipment is on the railroad or on the canal-boat. The costs of carting the packages to and from the freight depot and the wharf would in all likelihood not differ a great deal. The cost of moving from one station to another would probably be a little less by boat than by train. The cost of terminal expense may be relatively so high in both cases that the difference in cost of line haul may not be a factor of serious consideration. Regularity of service might probably be the chief matter of concern in the case. Both railroads and canals should devise systems of handling package freight at terminals in order to reduce the cost of shipment.

Package shipments in less than carload lots constitute, however, only a very small percentage of all freight business. The great bulk of railroad deliveries is in carload shipments. Where both the consignor and consignee have water-front facilities they could no doubt ship more cheaply by water than by rail; but not many industries are so situated. Coal, grain, and ore-handling seem to have been generally planned for such transportation in the Great Lakes region. Grain loaded from cars or lake boats at Buffalo, through elevators into barges on the canal and delivered through elevators at New York City into ocean-going ships could, no doubt, be handled more cheaply by the canal than by the railroad; and for that reason, most of the canal traffic will continue to be grain and similar bulk material.

An off-canal shipper can ordinarily find it just as cheap, or oftentimes cheaper, to ship by rail in the case of carload lots if the industry has its own siding and the expense of team haul is thus eliminated. The expense of terminal haul by truck or team is often an item of considerable magnitude compared with the line haul. It frequently happens that the expense of collecting and delivering the freight at the terminals is more than the entire line haul.

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† Received by the Secretary, November 26, 1926.

INTERSTATE WATER PROBLEMS AND THEIR SOLUTION

Discussion*

By M. C. Hinderlider and R. I. Meeker, Members, Am. Soc. C. E.†

M. C. Hinderlider‡ and R. I. Meeker,§ Members, Am. Soc. C. E. (by letter).‖—In the brief period that has elapsed since this paper was presented, many events have emphasized the growing importance and the advantages of the treaty or compact method as a means of solution of interstate river problems. The Colorado River problem is still an open question, both interstate and international. Flood control problems of interstate rivers of New Mexico, Oklahoma, and Texas have been under discussion, and an agreement has been drafted to be submitted to the Legislatures of the three States. The South Platte River Compact between Nebraska and Colorado has been ratified by Congress and is in operation. The Lower Rio Grande situation between Mexico and the United States is pressing for solution, and efforts are being made to entangle this river problem with the international phase of the Colorado River. The proposed Colorado River Aqueduct for Los Angeles, Calif., and associated municipalities, has taken definite form. An interstate commission is studying the Snake River for a compact or treaty among the States of Wyoming, Idaho, Washington, and Oregon. The consummation of a North Platte River Compact between Colorado, Nebraska, and Wyoming is looked for shortly, and interstate litigation is threatened on the Rio Grande.

The discussion¶ by State Engineer (now Governor) Emerson is a substantial contribution to the subject and touches on one additional phase of interstate river conflicts not covered by the writers.

General concurrence in the treaty method as a sane means for solving interstate river controversies is expressed by Mr. Emerson. Wyoming first tried the litigation method of settlement, and, like Colorado, found that method less satisfactory than agreement through friendly negotiations; this, too, despite the Laramie River decision of the U. S. Supreme Court, which is improperly referred to by many as completely wiping out State lines and as a victory decision for the lower State. Unfortunately, this erroneous

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* Discussion on the paper by M. C. Hinderlider and R. I. Meeker, Members, Am. Soc. C. E., continued from August, 1926, *Proceedings*.

† Authors' closure.

‡ State Engr. of Colorado, Denver, Colo.

§ Cons. Engr., Interstate River Compacts, State of Colorado, Denver, Colo.

‖ Received by the Secretary, December 6, 1926.

¶ *Proceedings*, Am. Soc. C. E., May, 1926, Papers and Discussions, p. 1026.

interpretation of the decision persists. In other words, some attorneys and engineers believe that the Laramie River decision can be applied in blanket form to interstate streams whereby all waters of a river basin may be administered as a unit, thereby resulting in a common administration of all priorities from the upper end of such river basin to the lower end, irrespective of State authority. Until this viewpoint is dispelled, many false assumptions will be injected into interstate river problems. Those who hold such views and look on the Laramie River decision as a panacea for interstate river problems should re-read and study the decision with care, and should read the decree as well as the opinion.

In the Wyoming case the Supreme Court did not declare that appropriations of individuals supersede State lines. When the opinion and decree are read together and are applied to the facts in that case, it is found that the Court decided that where each of two adjoining States recognizes the justice of the fundamental principles of prior appropriation in the distribution of water of its streams among its citizens, and a controversy arises between the two States respecting the use of the waters of a river common to both, the Court, as between the States, will apply the same general principle of justice in making an equitable distribution of the waters of the stream; and, after determining the average dependable supply of the stream, will so allocate or apportion such average dependable supply between the States (not between individual appropriators in the different States) in such a manner as to furnish a reasonably dependable supply of water for the irrigation of the land already irrigated in each State, and will apportion the surplus between the States for proposed new projects. The Court, however, qualified even this general rule by certain conditions unknown to the law of appropriation, namely, that the burden is upon each State to use every reasonable means to conserve the common water supply and, in a case such as the Laramie River, that the entire burden of providing storage is placed upon the lower State; that the water will be allocated to the States in bulk for interstate distribution by State officials; and that the State of origin (where storage facilities are limited) shall be entitled to a preferential right of annual diversion, irrespective of water supply and irrespective of the fact that such diversion is by the most junior appropriator of the entire stream system, and that such preferential diversions may interfere with the supply of the most senior appropriators, located in the lower State.

One of the writers (Mr. Meeker) was the Hydraulic Engineer for the State of Wyoming in that case, and was surprised to find that by the much heralded decision of "priority regardless of State lines", the Court had, in fact, decreed to the Laramie-Poudre Tunnel enterprise an absolute right to divert a given quantity of water each year, regardless of river conditions and in the face of the fact that, by the opinion, the tunnel enterprise was found to be the junior appropriator in both States.

When it is considered that this decision places the burden of river regulation and water storage entirely upon the lower State for the benefit of junior appropriators in an upper State, and when the other phases of the decision are considered in the light of the facts, there is little difference

between the principles applied in the Laramie River case and in the Kansas-Colorado decision. In both cases the Supreme Court equitably apportioned the waters of the river between the two States in a manner conforming to the fundamental principles of justice recognized within each State.

The Laramie River conflict was a concrete interstate water problem which did not contain all the seeds of conflict embraced in interstate issues of larger and longer rivers like the Colorado, North Platte, Rio Grande, and other major interstate river controversies. The decision, therefore, will have only a limited application to the larger interstate river problems, which, if litigated for solution, must rest on some additional features of interstate river uses.

Considerable space is devoted by Mr. Emerson to the North Platte River embargo imposed by the Federal Government, which resulted in the intolerable Pathfinder situation created in the North Platte Basin in Wyoming whereby "Carey Act segregations were refused, desert land entries were denied, rights of way across Government land were not granted, and, in other ways, developments which proposed a use of water in Wyoming from the North Platte River and its tributaries were blocked by the Federal Government" due to the supposition or fear of an inadequate water supply.

The North Platte embargo included also that portion of the North Platte Basin in Colorado known as "North Park" where the river has its source. Irrigation development in that area was also effectively withheld for a period of twenty years on the erroneous theory of insufficient water supply for Pathfinder Reservoir (capacity, 1 070 000 acre-ft.). Applications for rights of way for canals and reservoirs (of more than 1 000 acre-ft annual use) in Colorado over Government land were likewise refused. Construction of canals and reservoirs in North Park was stopped and, after protracted delays, this resulted in the destruction of the enterprises and total loss to the claimants. On one Carey Act project that was involved about twenty miles of canal actually had been constructed. Engineering studies of the water supply of the North Platte Basin, including studies of water requirements of irrigated and irrigable lands, consumptive use, return flow, and drainage recovery, show that the North Platte embargo was imposed without adequate knowledge of the water supply. The pathetic feature of the whole embargo program, however, is that the action of the Government was illegal and that private development was prevented or destroyed without legal or moral rights.

A parallel case occurred on the Rio Grande in Southern Colorado and Northern New Mexico, where a similar interstate river embargo, declared in 1907, was laid against the construction of reservoirs and canals involving the use of water in excess of 1 000 acre-ft. per year, on the erroneous theory that larger projects might interfere with the water supply of the Elephant Butte Reservoir (capacity, 2 638 000 acre-ft.), and with the treaty obligation of a mere 60 000 acre-ft. per year to Mexico. The Rio Grande embargo has been just as intolerable to Colorado as the North Platte embargo was to Wyoming, for needed reservoir development was prevented and projects were ultimately abandoned.

In May, 1925, the Rio Grande embargo was declared to be, and admitted to have always been, illegal and was cancelled by the Secretary of the Interior. Comprehensive engineering studies of water requirements and water consumption of the Rio Grande Basin in Colorado, New Mexico, and Texas (above Fort Quitman, 80 miles below El Paso), have also shown that the Rio Grande embargo was unwarranted. Surely projects in Colorado cannot be prejudiced in their relations with other lower river enterprises by the effects of an illegal embargo which constituted a cause of delay beyond their control.

The North Platte and Rio Grande embargoes have withheld development in four States for a period of twenty years. Although fathered at Washington and imposed by a Federal Bureau, they have been in fact measures favoring Lower Basin States, since they have effectively neutralized the strategic head-water position of Colorado and Wyoming on the North Platte, and Colorado and New Mexico on the Rio Grande.

In his excellent discussion Mr. Henny* points out a few of the many problems that have confronted those who have pioneered the field of settlement of interstate water problems. His suggestions are timely and valuable in developing the subject-matter on a broad basis.

It so happened that of the many Western States which will ultimately face the same problems, Colorado, by reason of her more rapid development, was the first to meet assertions of adverse water claims on behalf of neighboring States. Colorado was the first to be attacked and the first to be forced to defend. Many have entertained an erroneous view of the issues in both the Kansas-Colorado and the Wyoming-Colorado cases, and particularly the views of those who formulated the written pleadings in those cases. It is a rule well established that in all litigation each side asserts its most extreme doctrines and brings forward its maximum claims, and those who are prone to criticize the attitude of Colorado, as expressed in the pleadings in that case, should withhold such criticism until they have acquainted themselves with the extreme assertions set forth in the bills of complaint. That is particularly true in fairness to one of the States which brought suit against Colorado and now finds itself confronted with a situation similar to that occupied by Colorado, and so presents the same arguments advanced by Colorado in both the cases mentioned.

Unconsciously, Mr. Henny does injustice to Colorado and her people in his statement that the decision in the Wyoming case caused "a radical change in the attitude of Colorado", and that not until after that decision did Colorado begin to "point out the objections to Court proceedings and to advocate amicable negotiations and settlement by treaty, otherwise called the compact idea." There has been no change in Colorado's attitude.

The decision in the Wyoming case was announced June 5, 1922. The interstate treaty method of settlement of interstate water controversies was initiated in 1916; it was advocated and had been in process of development since 1911, by one of the attorneys who represented Colorado in the Wyoming

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1293.

case. The proceedings which resulted in the compact between Colorado and Nebraska concluded April 27, 1923, were commenced in 1916, and were authorized by the Legislature of Colorado in 1921, more than a year prior to the decision in the Wyoming case. The resolution adopted by representatives of the Colorado River States at Denver, Colo., on August 27, 1920, calling for the formation of the Colorado River Commission to negotiate the Colorado River Compact, was suggested by Colorado and drawn by the Colorado representative. The Colorado Legislature, with the Legislatures of the six other Colorado River States, authorized the formation of the Commission during the early part of 1921, and the Commissioner for each State had been appointed prior to May 10, 1921, when the Governors of the seven States met at Denver and formulated resolutions addressed to the President and the Congress requesting action by Congress for participation by the United States in the conclusion of the interstate compact. The Act of Congress was approved August 19, 1921, and Mr. Hoover was appointed as a Federal representative shortly thereafter. The Colorado River Commission had organized, had concluded public hearings, and was prepared to reconvene in executive and final session prior to the decision in the Wyoming case. The Colorado and New Mexico Legislatures authorized the La Plata River Commission in 1921, and the Commission was appointed and its deliberations were largely concluded during that year. The 1921 Session of the Colorado Legislature also authorized the appointment of a commissioner for Colorado to join with a similar commissioner for Kansas in the settlement of the pending interstate litigation on the Arkansas River, and authorized a similar commission to conclude an interstate compact between Colorado and Wyoming with respect to the waters of the Laramie River. The settlement of interstate river controversies by interstate compact was considered and discussed in the brief filed by the Colorado attorneys in the Wyoming case, and the methods of procedure in such matters had been under discussion since 1912.

Suggestion of the use of the treaty-making powers of the States, by consent of Congress, in the settlement of controversies respecting interstate rivers, was prompted by the early recognition of the futility of settling such matters by litigation, save as a last resort, and was earlier suggested by Colorado by reason of the fact that Colorado happened to be the first of the Western States with which the interstate water issue was drawn. Most of the Western States are in more or less the same position as that occupied by Colorado; even by 1920, they were not only urging the same arguments of State sovereignty theretofore advanced by Colorado, but were recognizing the necessity of prompt and effective measures in the settlement of title to interstate rivers as a preliminary to further major construction on which might be predicated monopolistic claims of "vested rights". The injustice of subjecting rights of States to the judgment of masters appointed by Federal Courts, wholly or largely uninformed of the subject matter litigated, recommended the treaty method of settlement and brought about the prompt action resulting in the conclusion of the Colorado River, La Plata River, and South Platte River Compacts.

Apparently from his discussion Mr. Henny entertains the view that so-called "vested rights" of individuals exist in rivers, which rights are so

protected by the Fifth Amendment to the Constitution that both Congress and the State Legislatures are powerless to provide for the equitable distribution of the use of the waters of interstate streams between the interested States—in other words, that the rights of a user of water in one State are greater than the rights of his State or of a neighboring State. The writers fail to comprehend how the rights of an appropriator in any State can be any greater than the rights of his State, the source of his title. The rights of his State may be limited by the rights of neighboring States, as determined either by interstate compact or by decision of the U. S. Supreme Court in a controversy between the two interested States, and certainly his rights would be defined by the limitations placed on the rights of his State. In other words, the rights of the State's grantee—the water user— at all times are subject to whatever limitations may be defined with respect to the relative rights of his State either by compact or Supreme Court decision. The grantee (the appropriator) can have no greater right than his State has power to convey. To urge that a water user has a right greater than that of his State, and derives his title from a source of power foreign to, independent of, and superior to, the jurisdiction of his State, is to challenge the autonomy of the State and to declare that each of the Western States was not admitted on an equality with the other States. It is true, there are advocates of the doctrine that all water rights are derived, not from the State, but from the United States, and that the disposition of the waters of the Western streams, and, perforce, of all the streams of all States admitted to the Union since the adoption of the Constitution, is in the keeping of Congress and wholly removed from State control save as permitted by Congress. Only by this doctrine could an appropriator pretend to claim rights superior to those of his State, or immune from the terms of any compact to which his State is a party and which shall have received the sanction of Congress. The writers hold Mr. Henny's opinions in too great respect to believe that his suggestions are founded on a doctrine so abhorrent to, and destructive of, the fundamental principles of the union of equal States of equal power which compose the Nation.

Suggestion is made of the need of a Federal representative on a compact commission to protect treaty rights of the Indians; also concerning the desirability of a representative from the Department of the Interior to protect inchoate water rights of Federal Reclamation projects.

Mr. Henny seems to overlook the fact that the negotiations involve the rights of sovereignties and are similar to international proceedings. The protection and safeguarding of such water rights are properly focused through the Federal representative who may have as many assistants and advisers as he desires. In negotiations of the Colorado River Compact the Federal Commissioner was not a representative of any Federal Bureau, but was a Presidential appointee selected for that specific task and was assisted by special counsel and engineers for the United States representing the Government projects, Indian obligations, international relations, navigation, and any other interest the Government may claim under the Constitution.

In any interstate treaty respecting the use of the waters in interstate rivers, the duties imposed on the river by international treaties, rights of

control of navigation, and other rights in the stream predicated on the surrender of powers by the States to the Federal Government, must be respected and, preferably, by representation on the Commission as well as by the action of Congress in approving the interstate compact. However, these rights are represented by different Federal executive branches. International rights are represented by the Department of State; rights of navigation by the Department of War; and Indian rights by the Department of the Interior. National reclamation, which by the terms of the Reclamation Act, as well as by the debates of Congress prior to the adoption of that Act, is at best but Federal aid of State development, and in which under the Act all proceedings must be in accordance with the laws of the States where the projects are constructed, is, for the time being, in the control of the Department of the Interior. Any of these several branches of Federal interest may be transferred from one Department to another by mere Act of Congress, and may be changed about from time to time and as frequently as Congress may determine.

It would appear that while it is advisable that a Federal representative participate in the work of any interstate river commission, such representative should not be from any one Department, or sub-branch, but should be one specially advised on the problems involved, representing the United States by Presidential appointment, and not merely some executive branch of Federal activities. This is well illustrated by the fact that while irrigation is at present uppermost in the minds of the American public, it probably will be overshadowed shortly by the development of electrical energy which is in charge of a separate Executive Department under the control of the Secretaries of War, Agriculture, and the Interior, while the Forestry Department, controlling the greater part of the National public lands, is a branch of the Department of Agriculture. It would appear that the promotion of interstate peace and comity is of sufficient importance and dignity to justify the appointment of a special representative by the United States to participate in the deliberations of each interstate river commission where Federal interests are in any manner involved. The negotiations are between sovereignties, each independent in its own sphere. It is hardly sufficient that a representative of some Bureau, or the substitute of some Executive Department, be "handy man" in such matters.

It has been suggested that some regional authority is required on an interstate river to execute either a Court decision or a compact. This may be desirable for some of the larger unsolved interstate river problems, but is not always necessary. No regional authority was designated in the Laramie River decision by the U. S. Supreme Court and none is required. The State Engineers of Colorado and Wyoming designated as the administrative authorities in control of State waters, naturally have charge of the execution of the Court decisions pertaining to water. In the La Plata and South Platte Compacts provisions are made for the State Engineers jointly to supervise and administer the compact obligations. The Colorado River Compact likewise provides for some administration supervision by the respective State

Engineers, the Director of the U. S. Reclamation Service, and the Director of the U. S. Geological Survey.

Concerning the statement that the compact method will not eliminate Court proceedings but may only lessen litigation—while isolated cases may occur which have to follow the legal route of solution, the writers believe that compact or treaty solutions will generally and practically eliminate Court proceedings.

A criticism has been offered to the effect that the compact method is cumbersome where six or seven States are involved on an interstate river. This is more true of litigation, for litigation compounds its delays and costs directly in proportion to the number of States involved. If eleven years are required for a Court decision in a two-State dispute, how many years would very likely be required to litigate a six or seven-State river dispute with Federal Reclamation projects and a foreign nation involved?

Mr. Henny refers to the Rio Grande international situation concerning the treaty with Mexico and the yearly obligation of 60 000 acre-ft., and comments as follows:

"The supply of the reservoir [Elephant Butte] is seriously threatened by contemplated storage in Colorado, and although the quantity assigned by the treaty with Mexico is small in relation to the full capacity of the reservoir, a series of dry years, such as has occurred in the past, may so deplete the reservoir as to render delivery of the stipulated quantity impossible."

The writers believe the latter view scarcely probable and impossible of substantiation. The international obligation is less than 3% of the average annual water production of the Rio Grande Basin above El Paso, and less than 8% of the average yearly flow of the Rio Grande originating in New Mexico above San Marcial at the head of the Elephant Butte Reservoir. The Elephant Butte Reservoir, with a capacity of 2 600 000 acre-ft., is the largest reservoir in the United States, and the capacity is two and one-half times the average yearly flow at San Marcial. However, it is a well-known fact that the mere seepage and return waters from irrigation of lands between Elephant Butte and El Paso are far more than adequate to insure delivery of water under the International Treaty with Mexico, irrespective of drought on the head-waters of the Rio Grande. It appears that, in the effort to make the water supply for the Elephant Butte doubly secure, at the expense of needed reclamation in the upper States, the insignificant quantity guaranteed Mexico under the International Treaty has been overworked until the situation is much like the proverbial tail wagging the dog. Going a step farther, the water supply of the Elephant Butte Reservoir is not seriously threatened by the proposed projects on the Middle Rio Grande Valley at Albuquerque, on the Puerco, on the Santa Cruz near Espanola (all in New Mexico), or by the proposed reservoir construction on the head-waters in Colorado. Practically all these proposed developments are to aid lands now irrigated, or to make possible the drainage of lands now water-logged and wasting large quantities of water through evaporation. Such lands were irrigated before the Elephant Butte Reservoir was contemplated. Drainage recovery of water now wasted by evaporation will be equivalent to an increased water supply in the river.

The upper river structures in both New Mexico and Colorado will assure drainage of the upper irrigated areas, will regulate river flows, will furnish clear water to the Middle Basin, and will assist in decreasing the silt menace with which the Elephant Butte Reservoir is confronted, namely, about 20 000 acre-ft. of silt deposition per year.

When considering interstate river problems, engineers have been prone to overlook certain fundamental jurisdictional phases. Among these are: That the question arises between States, possessing all the sovereignty of independent nations, except for those powers and that limited jurisdiction surrendered by each State to the United States; that principles of international law are involved; that claims of foreign servitude for the benefit of one State are made over the territory of another State, without its consent; that such claims are abhorrent to principles of international law; that assertions of right are made which, if sustained, would destroy the equality of the States and the autonomy of one State for the exclusive benefit of another State (or its citizens) without the consent of the former and without compensation for the property taken; that water is necessary to the life and preservation of each of the arid States, and the right to its use must be protected in justifiable self-defense; that the injection of principles of Federal control or interference with State streams under assumption of powers not granted by the Constitution or predicated on theories calling for conclusions contrary to the intent of the Constitution, whether prompted by motives of temporary expediency or desire to over-ride State jurisdiction, only tend to destroy interstate comity and to bring the States to a position of resistance; that all doctrines of water law are, after all, but rules of local administration by which a sovereign—the State—regulates the use of its natural resources among its nationals, and are not applicable to interstate problems because of lack of jurisdiction to create machinery to enforce such rules of administration; and that to attempt to effect such administration would be wholly to supersede all State authority over a resource which is essential to the preservation of the State and which must ever remain subject to local eminent domain in order that the State may re-adjust the use of its life-giving fluid to the ever-changing necessities of its people.

When these and many other factors are considered in their true light, and with a desire to protect the autonomy of the States for the necessary preservation of the Nation, which is but a Union of otherwise independent States, the whole subject is taken out of the plane of ordinary local rules and thought, and must be approached from a much broader standpoint than the mere protection of selfish individual property rights within a State. This the interstate compact method seeks to accomplish by bringing about contractual relations between sovereignties with full realization of the sanctity of treaties and the necessity of their fulfillment without the interposition of external annoyances and intermeddling, at all times having due regard to the rights granted the United States by the Constitution and the protection of these rights by the contracting States.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

AUGUSTINE LEE DABNEY, M. Am. Soc. C. E.*

DIED AUGUST 23, 1926.

Augustine Lee Dabney was born on November 17, 1870, at Vicksburg, Miss. During the summer of 1886 he served as Rodman on levee construction work and on railroad preliminary surveys, and in the summers of 1887 and 1888 he worked on levee construction as Instrumentman and on railroad location, resigning this position to prepare for college.

He attended Washington and Lee University, at Lexington, Va., during the session of 1888-89, and the following summer became First Assistant Engineer on levee construction, Topographer, and, later, Precise Recorder in the United States Survey of the Red River, Louisiana. In 1890 he served as Resident Engineer on the Yazoo-Mississippi Delta Levee District in charge of constructing five miles of new and enlarging four miles of existing levees.

Mr. Dabney returned to Washington and Lee University for the sessions of 1890-91 and 1891-92 and was graduated in 1892 with the degree of Civil Engineer and high rank in a large class.

From June, 1892, until May, 1896, he was Assistant United States Engineer in charge of all Federal work in the Lower White River Levee District, directing the prosecution of work costing about $400 000 and having full charge of high-water and flood protection. He made locations and estimates for new levee lines and passed on the completed work which he did with such success that he was promoted to be Assistant Chief Engineer of the Yazoo-Mississippi Delta District, in which capacity he served from 1897 to 1902. Many of the wonderful works planned and carried out there stand as a monument to him.

In February, 1902, Mr. Dabney left the Government work, engaging in private practice, but after successfully carrying out several important projects of sewage disposal, drainage, and reservoir engineering, he was again called to serve the Government, this time as Assistant Chief Engineer of the Tallahatchie Drainage District, comprising 1 150 000 acres of the Yazoo-Mississippi Delta. After giving two more years to this work, he again entered the more profitable field of private practice as a specialist and acknowledged authority on drainage, flood protection, sewage disposal, and related engineering work.

In 1917 he entered the Military Training Camp at Fort Oglethorpe, Ga., and was commissioned a Major of Engineers, United States Reserve Corps, in which capacity he did signal service in designing cantonment sewers and supervising such construction.

* Memoir prepared by Arthur J. Dyer, M. Am. Soc. C. E.

At the close of 1917 Major Dabney investigated and reported flood protection for the Chattanooga, Tenn., District—a project variously estimated at between $3 500 000 and $6 500 000 and, in 1918 and 1919, he held the position of Engineer in Charge of Water-Works, Sewerage, and Drainage, at Muscle Shoals, Alabama, the big nitrate plant. After the World War he returned to private practice at Memphis, Tenn., in which he was successfully engaged until his death.

During his long and useful career Major Dabney was the author of many technical publications well and favorably known to engineers and others interested in the subjects in which he specialized. These are generally recognized as valuable and authoritative contributions to this important branch of the profession among others being a noteworthy treatise on the relation between precipitation, run-off, and discharge.

Major Dabney was a member of the Illinois Society of Engineers and the National Geographic Society, and of the Engineers' Club, the City Club, and Chamber of Commerce of Memphis, Tenn., in which city he made his home and practiced his profession during the latter years of his life.

He was a gentleman of the highest order of intelligence and of the strictest integrity; a man of fine scientific attainment combined with not only highly specialized, but well balanced practical knowledge and experience, which, with unusually accurate and impartial judgment, pre-eminently fitted him for membership on the Tennessee State Board of Architectural and Engineering Examiners, on which Board he served conspicuously as Chairman. In his death the Engineering Profession has lost a useful member and the community in which he lived an excellent and eminent citizen.

Major Dabney was elected a Member of the American Society of Civil Engineers on October 2, 1901.

THOMAS ELMER PHIPPS, M. Am. Soc. C. E.*

Died February 22, 1926.

Thomas Elmer Phipps was born near Janesville, Ill., on May 18, 1879, the son of Napoleon and Hannah Phipps. After completing his studies in the public schools Mr. Phipps taught school in Coles and Cumberland Counties, Illinois. In 1910 at Valparaiso, Ind., he received a Bachelor of Science degree from Valparaiso College, and after his graduation, became Principal of the Public Schools at Lerna, Ill. In 1903 Mr. Phipps entered the University of Illinois and completed the four-year course in 1906, graduating with high honors.

During the vacation periods he was employed as Instrumentman on the Cairo Division of the Cleveland, Cincinnati, Chicago and St. Louis Railway. After graduating from the University until February, 1907, he was Resident Engineer of the Division, in charge of forty-two miles of reconstruction work.

* Memoir prepared by Frank F. Sinks, M. Am. Soc. C. E.

During the summer of 1907, Mr. Phipps became Superintendent of the Walsh Construction Company, in charge of steam-shovel crews, reconstructing and double-tracking the main line of the Cleveland, Cincinnati, Chicago and St. Louis Railway between Terre Haute and Indianapolis, Ind. On the completion of this work he went to Seattle, Wash., where he entered the City Engineer's Office, remaining until April, 1908.

Mr. Phipps then entered the service of the Chicago, Milwaukee, and St. Paul Railway Company and was in its Engineering Department until October, 1914. During this time he was for two years Chief Draftsman in the Division Office at Ellensburg, Wash., and one year in charge of the construction of fourteen miles of the mountain end of the Everett Line. From November, 1911, to October, 1914, he served as Assistant Engineer in charge of locating parties on the Olympic Peninsula; of a survey of the Bellingham Northern Railway; of investigations and locations of transfer landings on Puget Sound; of building landings and terminal tracks in Bellingham; and of maintenance work.

In October, 1914, he became Assistant Engineer for the Public Service Commission of the State of Washington and Chief Engineer of the Commission in February, 1915. This Commission has jurisdiction over the valuation, rates, and service regulations of all public utilities in the State of Washington. Among the more notable causes in which Mr. Phipps as Chief Engineer prepared the evidence were State-wide valuation, rate study, and promulgation of operating rules for the Pacific Telephone and Telegraph Company; valuation and rate study of the Seattle Lighting Company; valuation of the street railway, light, and power systems of the Puget Sound Traction, Light and Power Company; and valuation and rate study of the Puget Sound Navigation Company.

During the latter part of 1917 Mr. Phipps was commissioned a Captain in the Engineer Corps. He was called into training at Camp Lee, Petersburg, Va., on January 1, 1918, at which time he resigned his position as Chief Engineer of the Public Service Commission of the State of Washington. Due to defective hearing he was not permitted to go overseas. In March, 1918, Captain Phipps was detailed to duty at Governors Island, New York, as Assistant to the Engineer Officer in charge of construction under the New York Depot Quartermaster. In April he was put in charge of all construction work on Governors Island as well as work of like character in New York City and vicinity. The construction work on Governors Island consisted of a large number of warehouses, including sprinkling systems, railroad tracks and yards, locomotive roundhouse, a ferry transfer bridge, a system of mains, stand-pipes, and pumping station for fire protection, fresh-water supply, and all other work necessary for a military depot. In addition to the work on Governors Island, Captain Phipps had charge of the repairs and alterations of many buildings in New York City and vicinity that were used by the Depot Quartermaster for various purposes. One who was closely associated with him at Governors Island says:

"Captain Phipps proved himself a very able engineer. His work was well systematized and his judgment with unusually good common sense were

strong factors in the results obtained. He was forceful in his work and eminently fair to everybody, but was always careful to see that the Government received its due in labor and materials furnished. His work was a credit to himself and to the Engineer Corps."

Captain Phipps was mustered out of the Army on February 5, 1919, and entered private practice in Seattle, specializing in public utility valuation and rates. In 1923 the City of Seattle employed him as its Engineer in contesting the proposed rate increase of the Pacific Telephone and Telegraph Company. In 1924 the City of Seattle again employed him when the Seattle Lighting Company reduced its British thermal unit standard of gas. The Puget Sound Telephone Company, a pioneer in the use of time-measuring devices to base rates for telephone services, became involved in a controversy, and employed him as its expert Engineer. He also made studies of rates for a number of public utility corporations operating in the State of Washington and in British Columbia.

In the latter part of 1925, he was employed by the Tax Commission of the State of Washington, but due to failing health was unable to render much service and was obliged to resign.

On February 13, 1913, Mr. Phipps was married in Seattle, to Mrs. Jennie C. Trimnell, who, with one child, Jane Evelyn, survives him.

Mr. Phipps was known among his associates as a man having the highest ideals of professional ethics, of sterling integrity, and a staunch friend.

He was a Mason, having been a member of the Blue Lodge, Scottish Rite, and Shrine. He was also a member of the Seattle College Club.

Mr. Phipps was elected an Associate Member of the American Society of Civil Engineers on October 3, 1911, and a Member on November 10, 1915.

WALTER AUGUSTUS SUMNER, M. Am. Soc. C. E.*

DIED JUNE 25, 1926.

Walter Augustus Sumner was born in Unionville, Mo., on December 17, 1884, the son of Horace Augustus and Ida May (Holbrook) Sumner. He was the youngest of three sons, and was educated in the grammar school and in the East Denver High School after his parents had taken up their residence in Denver, Colo. in 1886. He entered Leland Stanford Jr. University, in California, in 1904, remaining in that institution until the earthquake in 1906. He was made a member of the Sigma Alpha Epsilon Fraternity while at Stanford.

Following the severance of his connection with Stanford University, and until the latter part of 1908, Mr. Sumner was employed on several railroad surveys, as Chainman, Rodman, and Levelman, principally as Levelman on the Denver, Northwestern and Pacific Railway (now the Denver and Salt Lake Railway), under the direction of his father, H. A. Sumner, M. Am. Soc. C. E., who was Chief Engineer of the Company.

* Memoir prepared by Horace A. Sumner, M. Am. Soc. C. E.

During 1909, he served as Locating Engineer on the Green River Canal Project in Wyoming, defining the alignment and grades of a canal 72 miles in length designed to carry 300 sec.-ft. of water. In 1909, Mr. Sumner was also Assistant Engineer on the location of a revised alignment of the Denver and Rio Grande Railroad from Soldier Summit, Utah, westerly, where a 2% grade was adopted to take the place of the 4% grade then in operation. This work was under the direction of James G. Gwyn, Chief Engineer.

From the latter part of 1909 until the summer of 1915 he was employed by the late D. G. Thomas, M. Am. Soc. C. E., Chief Engineer and General Superintendent of the Denver Union Water Company, as Engineer and Superintendent in charge of properties of that Corporation outside the City of Denver. His duties involved the construction, repair, and maintenance of a water plant and system supplying Denver with a maximum of 87 000 000 gal. per day of filtered and treated water. A thorough knowledge of the construction and operation of slow sand, mechanical, and rapid sand filters was acquired by him in this position, as well as a knowledge of the sterilization of filtered water.

During the period of Mr. Sumner's connection with this water system the construction of one diversion dam and sand trap in the bed of the South Platte River, and the installation of eleven miles of cast-iron, steel, and wood-stave pipe conduits up to 60 in. in diameter, were a part of the construction work under his charge.

In 1915 and 1916, he was engaged in research work connected with water sterilization, and in the development of a product known as "Sterilall".

During a part of 1916 he was employed by G. W. Harris, M. Am. Soc. C. E., Chief Engineer of the Coast Lines of the Atchison, Topeka and Santa Fé Railway Company, near Los Angeles, Calif., in charge of the construction of plain and reinforced concrete structures, and on matters connected with valuation of the railway. During the latter part of that year and the first half of 1917 Mr. Sumner was also employed by Mr. Harris to superintend the construction of a reinforced concrete multiple-arch dam, 150 ft. high, near Escondido, Calif., with 10 miles of 42-in. concrete distributing conduits.

Mr. Sumner took part in the World War, being commissioned by the War Department as First Lieutenant of the Engineer Reserve Corps on July 31, 1917. On December 28, 1917, he was ordered into service and received engineering training at Camp Lee, Petersburg, Va., where he remained about two months. He was then ordered to report to the officer in charge of the Cantonment Division, Quartermaster Corps, Washington, D. C., which later became the Construction Division of the Army.

After remaining in Washington until May 18, 1918, Lieutenant Sumner was detailed to supervise water and sewerage plants and the installation of plumbing and steam heating at Camp Perry Proving Grounds, Fort Clinton, Ohio. He also performed the duties of "Disbursing and Property Officer". While stationed at Camp Perry he received his promotion as Captain, Quartermaster Corps, U. S. Army, his commission bearing the date of August 24, 1918.

On September 14, 1918, he was ordered to report to Willoughby, Ohio, where he took station as Constructing Officer in charge of the construction of barracks and mess halls, with a hospital unit to accommodate the housing of 1 000 men engaged in the Chemical Warfare Service.

On the completion of this work, on November 21, 1918, he again reported to Camp Perry Proving Grounds, where he resumed his duties as Assistant to the Constructing Quartermaster. He was relieved of these duties on August 12, 1919, being then ordered to Peoria, Ill., where he was Constructing Quartermaster in charge of work at the Holt Manufacturing Company. He remained in Peoria until November 18, 1919, when he was ordered to duty as Constructing Quartermaster of the Picatinny Arsenal, near Dover, N. J., his work consisting of the construction of eleven additional buildings for the manufacture and storage of high explosives.

On September 25, 1920, Captain Sumner received his honorable discharge from the Army. It was a source of keen disappointment to him that he was not permitted to serve his country on foreign soil, but the wisdom of his retention on this side was proved by the valuable service which his engineering training rendered him capable of performing in lines of work so briefly described herein. His true patriotism was clearly shown by the cheerful spirit in which he bent his energies to the supply of the sinews of war to the comparatively few engaged in actual combat.

During a part of 1920 and until the middle of 1922, Mr. Sumner was engaged in private practice in New York, N. Y. Most of this time was given to the design of structures and to a general layout plan for the Fulton Lumber Company Terminal near Hoboken, N. J.

In 1922, and until April, 1924, he was engaged in consulting engineering and general contracting activities in Los Angeles and San Francisco, Calif.

From April, 1924, to August, 1925, he was employed by the Denver Municipal Water-Works, under the direction of R. S. Sumner, M. Am. Soc. C. E., General Manager, as Constructing Engineer in charge of the construction of a new and modern rapid sand filter plant at Marston Lake, near Denver. The capacity of this plant assures the filtration of a maximum of 64 000 000 gal. of water per day, and its cost was in the neighborhood of $2 000 000. Since the completion of that work he was engaged to the time of his death in general practice as Consulting Engineer and Contractor in Denver.

Mr. Sumner was adapted by nature for the profession of an engineer. He was gifted with a mind which prompted a close scrutiny into every detail of the work in hand, and made him conversant with the reasons for or against any mode of procedure. He was especially gifted with the faculty of handling men and attending to the assembling of materials to be used, thus avoiding unnecessary delays and extra costs which follows so often in works of magnitude.

Of a genial and happy disposition, he was a stimulus to all employees under him, and was able to control and carry on the work in hand with the loyal support of all members of his staff. Had he lived longer, the inherent gifts with which he was endowed and the further experience in professional

activities would undoubtedly have placed him among the brilliant and useful members of the Engineering Profession.

Socially he had an extensive acquaintance and his inborn wit and good nature were the delight of any gathering. He was married on October 27, 1909, to Jessie May Tucker, daughter of Mr. and Mrs. Cromwell Tucker, of Denver, Colo. His home life was happy, one which could be envied by all.

Mr. Sumner was elected a Member of the American Society of Civil Engineers on January 17, 1921.

ROSWELL JAMES AYDLOTTE, Assoc. M. Am. Soc. C. E.*

Died February 22, 1925

Roswell James Aydlotte, the son of John K. R. and Cora Aydlotte, was born at Chester, Pa., on March 9, 1892. He received his early education at the Chester High School and also at Bucknell University, Lewisburg, Pa. In September, 1910, he entered Drexel Institute, Philadelphia, Pa., where he took the Civil Engineering Course. He was graduated from that institution in June, 1914.

Mr. Aydlotte's engineering work began shortly after his graduation from Drexel Institute and led into many lines of the profession. In 1914-1915 he served as Surveyor in Class 1 of the United States Engineering Department, acting as Inspector of Construction, and in 1915-1916 he was engaged on the construction of sewers and highways in Oaklyn, N. J. In 1916-1917 he was employed on construction work at the Pennsylvania and New Jersey Shipbuilding Yards and in 1917 he was also engaged as Engineer in Hydraulic Dredging by the Delaware Dredging Company.

In 1917-1918 Mr. Aydlotte served as Chief Night Inspector of Ways and Piers at Hog Island, American International Shipbuilding Corporation and in 1918 his work took him to Boston, Mass., where he was Engineer for a contracting company on steam shovel work. During 1918-1919 he was employed as Engineer on the construction of a gravel-washing plant at Curtis Bay, Md., by the Arundel Shipbuilding Company.

In 1919 the Delaware River Dredging Company made use of his services in repairing the break in the sea-wall at Port Penn, Del., and in 1919 and 1920 he was a member of the firm of Aydlotte and Roop, Engineers and Surveyors, with offices in Philadelphia. In 1921 Mr. Aydlotte was appointed County Engineer of Delaware County, Pennsylvania, which position he occupied until the time of his death on February 22, 1925.

The most important phase of Mr. Aydlotte's engineering career began with his appointment as County Engineer of Delaware County. The office had been established only a short time before this, and Mr. Aydlotte was compelled to organize it on a large scale and get the force in condition for an active campaign of bridge replacement. The design of bridges in Delaware County had been done previously by engineers in private practice. Bridges

* Memoir prepared by C. M. Broomall, Assoc. M. Am. Soc. C. E.

had been constructed in a desultory fashion and the whole County was covered with structures that were never designed to carry the heavy loads imposed by modern traffic. Many of these bridges were dangerous either from overload or age. The situation was acute, and under Mr. Aydlotte's supervision the County Commissioners began the reconstruction of practically every bridge in the County.

It was in this work that Mr. Aydlotte showed his ability both as an engineer and as an executive. In the four short years from the time of his appointment as County Engineer until his death, he designed and superintended the construction of approximately thirty-two bridges, mostly of the reinforced concrete slab or arch type. His last work was the design and supervision of the beautiful Memorial Bridge, a two-arch reinforced concrete span of imposing dimensions, on the Philadelphia and Baltimore Pike, over Crum Creek. This bridge was completed before Mr. Aydlotte's death, although he did not live to receive the tributes which were accorded to him at its dedication ceremonies. The responsibility and exacting work connected with the building of so many bridges in so short a time no doubt played a part in hastening his untimely death.

His pleasant personality endeared him to his many friends and his energy and ability as an engineer are attested by his record.

On December 15, 1915, Mr. Aydlotte was married to Edith Ward. One child, Margaret Jane Aydlotte, was born of this marriage.

Mr. Aydlotte was elected an Associate Member of the American Society of Civil Engineers on October 2, 1922.

BERNARD HOOE FOWLE, Jr., Assoc. M. Am. Soc. C. E.*

Died April 23, 1926.

Bernard Hooe Fowle, Jr., was born in Spokane, Wash., on August 10, 1890. He was the eldest son of Bernard Hooe and Milly Dorsey Fowle. Through his father he was related by blood to the Washington, Ball, Chichester, Hooe, and Briscoe families, and was directly descended through his mother from George Mason, of Gunston, the author of "The Bill of Rights", and from Chief Justices Chew, of Pennsylvania, and Dorsey, of Maryland. He was related to the Dallas and Bush families, of Pennsylvania, and the Carrol, Murray, and Howard families, of Maryland.

His first ten years were spent in Spokane. He then moved with his family to Washington, D. C., where he attended private schools, entering Virginia Polytechnic Institute, Blacksburg, Va., in 1909. Mr. Fowle's four years at the Institute were distinguished by high scholastic standing. He was graduated as First Lieutenant and Adjutant in June, 1913, receiving a letter of thanks for his fearless and efficient performance of duty as a cadet officer, his actions having manifested a high order of moral courage.

* Memoir prepared by R. M. Miller, M. Am. Soc. C. E.

On graduation Mr. Fowle became a Draftsman for the Clinchfield Coal Corporation of West Virginia. In March, 1914, he transferred his activities to highway construction in Mercer County, West Virginia, as Resident Engineer. In November, 1915, he became Engineer for Mr. John L. Vaughan, at Petersburg, Va., on various development projects and electric interurban railway work. In May, 1916, he entered the service of the Virginian Railway Company as Instrumentman and later was promoted to the position of Resident Engineer.

In May, 1918, Mr. Fowle left the service of the Virginian Railway Company to take charge of a residency for the Chesapeake and Ohio Railway Company. He entered the Army as Second Lieutenant of Engineers in July, 1918. As such, he served as Company Commander of "C" Company, 70th Battalion of Engineers, Fort Douglas, Utah, and, later, as Topographical Officer. The Battalion was ordered overseas, but reached the port of embarkation on Armistice Day.

In December, 1918, he was honorably discharged from the Army and returned to Utah where he served with the State Road Commission as Locating Engineer and, later, as District Engineer of the Southeastern and Southwestern Districts. In September, 1920, he returned to the service of the Virginian Railway Company, acting as Resident Engineer on several construction projects and, later, as Assistant Engineer with the Valuation Department. A slipshod piece of work by Mr. Fowle was unknown.

It should be added that Mr. Fowle was hard of hearing. The reason for his acceptance by the Army has never been explained, but it was doubtless due to a persistence and determination that considered no discouragement nor took into account repeated refusals. Such a handicap to any man expecting to see active service at the front required a degree of courage possessed by few. To him the difficult part was that played by the Medical Corps and not that played by the enemy.

Bred in the fiber of this man was the knowledge of the amenities drawn from a long line of high-minded people. The practice of the courtesies was to him as easy and natural as breathing. Those who knew him in war times and in civil life have learned much of those things that make of life something very much worth while.

Endowed with a brilliant mind that had the ability to grasp quickly and retain in usable form impressions gained from inspection or study, Mr. Fowle's career was, indeed, cut far too short, but the extent of the influence of such a life is without measure. Endowed also with high moral and physical courage, without a flaw, it can be said of him, "verily, he was a man."

On March 12, 1921, he was married to Lucille White, daughter of Mr. and Mrs. J. W. White, of Princeton, W. Va., who, with two children, survives him.

Mr. Fowle was elected an Associate Member of the American Society of Civil Engineers on July 6, 1920.